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Approximately half of the Corps 269 navigation lock chambers were built prior to 1940. Consequently, the concrete in these structures does not contain intentionally entrained air and is therefore susceptible to deterioration by freezing and thawing. Since more than three-fourths of these older structures are located in the Corps North Central and Ohio River Divisions, areas of relatively severe climatic exposure, it is not surprising that the concrete in many of these structures exhibits significant freeze-thaw deterioration. Depending upon exposure conditions, depths of concrete deterioration can range from surface scaling to several feet.

The general approach in lock wall rehabilitation has been to remove the deteriorated concrete and replace it with concrete or shotcrete. Explosive blasting has been used successfully at several Corps projects and appears to be the most cost-effective and expedient

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means for removing large quantities of concrete. Once the deteriorated concrete has been removed, conventional cast-in-place concrete has been used as the replacement material in most lock wall rehabilitation projects. Other replacement systems that have been used or proposed include shotcrete, preplaced-aggregate concrete, and precast concrete stay-in-place forms. In addition, several materials including latex-modified concrete, polymer mortars and grouts, conventional shotcrete, and latex-modified, fiber-reinforced shotcrete have been used as thin overlays on existing lock walls. Applications of the various rehabilitation systems and their performance to date are described in selected case histories.

PREFACE

The study reported herein was authorized by Headquarters, US Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32273, "Rehabilitation of Navigation Locks," for which Mr. James E. McDonald is Principal Investigator. This work unit is part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program sponsored by HQUSACE. The Overview Committee at HQUSACE for the REMR Research Program consists of Mr. James E. Crews, Mr. Bruce L. McCartney, and Dr. Tony C. Liu. Technical Monitor for this study was Dr. Liu.

The study was performed at the US Army Engineer Waterways Experiment Station (WES) under the general supervision of Mr. Bryant Mather, Chief, Structures Laboratory (SL), and Mr. John M. Scanlon, Chief, Concrete Technology Division (CTD), and under the direct supervision of Mr. James E. McDonald, Research Civil Engineer, (CTD), who prepared the report. Program Manager for REMR is Mr. William F. McCleese, CTD.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
bag (94 lb mass)	42.6377	kilograms
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or kelvins*
feet	0.3048	metres
gallons (US liquid)	3.785412	litres
horsepower (electric)	0.746	kilowatts
inches	25.4	millimetres
kips (force)	4.448222	kilonewtons
kips (force) per square inch	6894.757	kilopascals
miles (US statute)	1.609347	kilometres
ounces (avoirdupois)	0.02834952	kilograms
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
tons (2,000 lb mass)	907.18464	kilograms

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

REHABILITATION OF NAVIGATION LOCK WALLS: CASE HISTORIES

PART I: INTRODUCTION

Background

1. The Corps of Engineers currently owns and operates 269 navigation lock chambers at 232 project sites along inland waterways. Approximately half of these lock chambers were built prior to 1940. The average age of these 133 older lock chambers is nearly 67 years or well beyond the 50-year design service life. Since these structures were built prior to 1940, the concrete does not contain intentionally entrained air and is therefore susceptible to deterioration by freezing and thawing. Since the majority (78 percent) of these older lock chambers are located in the Corps North Central and Ohio River Divisions, areas of relatively severe climatic exposure, it is not surprising that the concrete in most structures exhibits significant deterioration.

2. A few of these structures have been rehabilitated with typical costs in the range of $10-20 million. The general approach in navigation lock rehabilitation has been to treat the projects as new work. Procedures followed for concrete operations have been those with which designers and contractors are familiar from past experience on new construction. However, there is increasing evidence that rehabilitation work is often more complex and that normal new construction procedures often do not produce satisfactory results in rehabilitation work. Obviously, in a rehabilitation program which could ultimately cost more than $2 billion there is a need for development of appropriate technology to ensure optimum utilization of available resources.

Purpose

3. The objective of this study was to develop, review, and analyze selected case histories involving rehabilitation of navigation lock walls.

Scope

4. Information on the rehabilitation of navigation lock walls was obtained through (a) review of periodic inspection reports, (b) visits to

project sites, (c) discussion with project personnel, and (d) discussion with designers and contractors. Although the information obtained from the various sources varied widely from project to project, attempts were made to obtain (a) a description of the project, (b) the cause and extent of concrete deterioration, (c) descriptions of rehabilitation materials and procedures, (d) rehabilitation costs, and (e) performance to date of the rehabilitated lock walls. Based on a review and analysis of the information obtained, recommendations for future rehabilitation were developed and areas which could benefit from research were identified.

PART II: CASE HISTORIES

5. Sufficient information to prepare a case history was obtained for 10 lock wall rehabilitation projects. Descriptions of the repairs, which spanned a period of approximately 35 years, are arranged in roughly chronological order in the following.

Lock No. 5, Monongahela River

6. Lock No. 5 was located at Brownsville, Pennsylvania, about 55 miles* above the mouth of the Monongahela at Pittsburgh, Pennsylvania. The lock had twin chambers each 56 ft wide and 360 ft long (Figure 1) with a lift of approximately 12.4 ft between normal upper and lower pools. The locks were constructed during the period 1907-1910 and received normal maintenance during the years following construction.

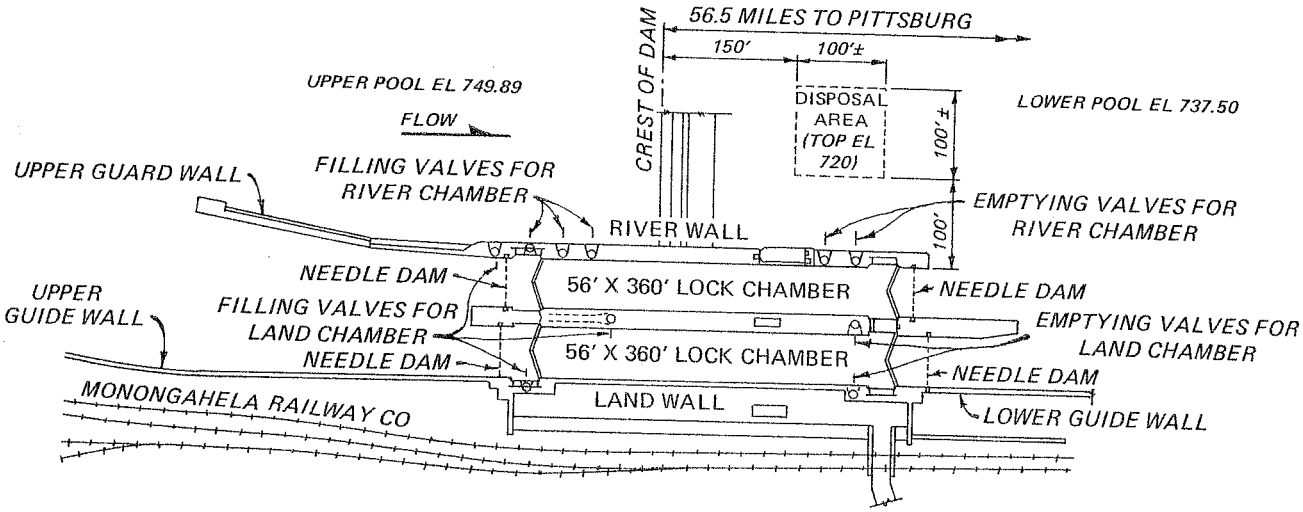

Figure 1. General layout of Lock No. 5, Monongahela River

7. By 1949, concrete deterioration had progressed to the point that refacing of the lock walls was required. Plans for the repair called for the removal of approximately 18 in. of old concrete from an area extending from the top of the lock walls to about 18 in. below normal pool elevation and the refacing of this area with reinforced concrete. In addition, the contract

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

included such incidental work as the furnishing and installation of structural wall armor and new corner protection, and the removal and replacement of existing corner protection, check posts, ladder rungs, hand railing, and gate anchorages. The specifications provided that the replacement concrete could be either conventional concrete or preplaced-aggregate concrete (Minnotte 1952).

8. Four bids ranging from approximately $85,000 to $259,000 were received. The low bid was submitted by Intrusion-Prepakt Company, Cleveland, Ohio, to whom the contract was awarded. The low bid was almost $60,000 lower than the second lowest bid. The successful bidder elected to use preplaced-aggregate concrete.

9. The contractor was required to keep one of the two chambers open to navigation at all times. The contractor elected to perform the work without constructing cofferdams, a procedure permitted by the specifications. This method necessitated the removal of existing concrete below pool elevation "in the wet," and required that the new concrete below pool elevation be placed behind watertight bulkheads to exclude pool water from the spaces to be filled with concrete.

10. The existing concrete was removed by line drilling the vertical face with holes spaced 6 in. on centers. Line drilling of the lower limit of the concrete to be removed was not specified, but the lower surface was required to be reasonably level. Light blasting for removal of existing concrete was permitted. Concrete removed from the lock walls was retrieved from the lock chambers and transported by barge to a designated disposal area.

11. After the old concrete had been removed and the face scaled, preparations were made for placing the mesh reinforcing and constructing the forms for the new concrete. Anchor bar holes, spaced 3 ft 4 in. vertically and horizontally, were drilled in the concrete to a depth of 2 ft 0 in. The holes were drilled on a slight downward slope to facilitate grouting. Wedge anchor bars were then inserted into the drilled holes and driven up on the wedges until the ends of the bars were fully expanded against the sides of the holes (Figure 2(a)). A grout mixture, composed of one part portland cement and two parts fine sand, with sufficient water to produce a plastic mixture, was then rodded into the holes around the bars. The grout was allowed to set at least 72 hr before being loaded in any manner.

12. Anchor bars were of two types. One type, 3/4 in. round, was hooked

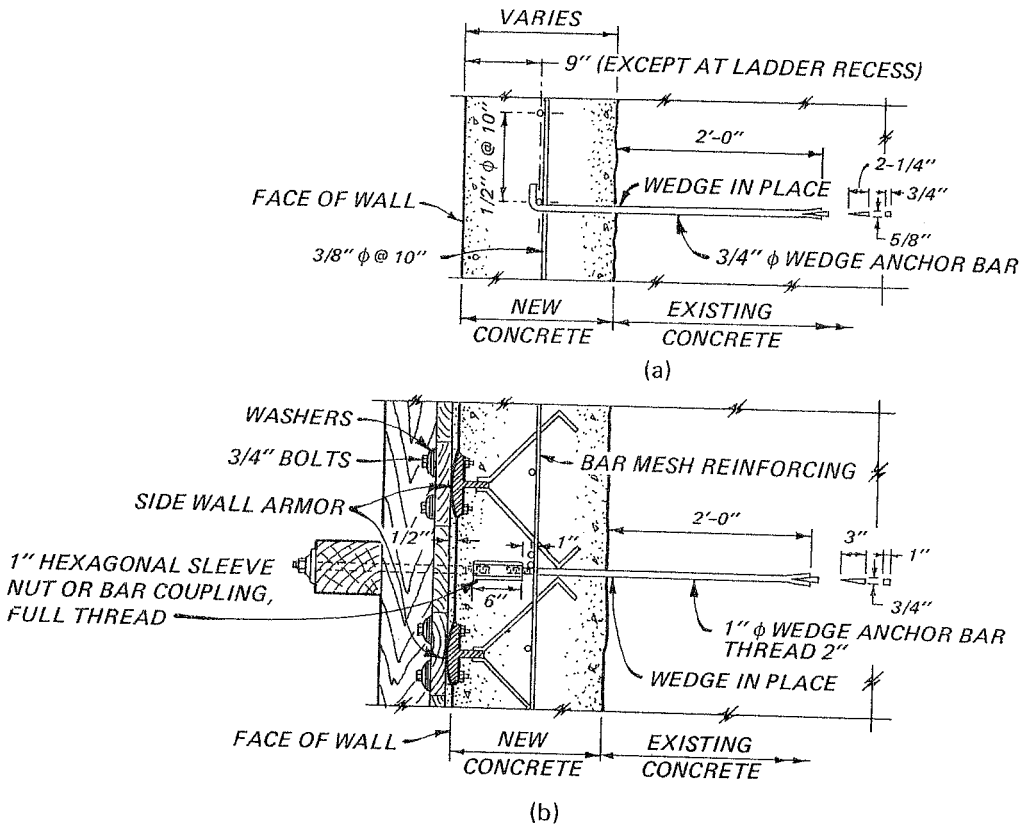


Figure 2. Repair details, Lock No. 5, Monongahela River

at the end and used to carry the reinforcing mesh. The mesh was spot welded to the anchor bars at a point 9 in. from the face of the form. The other type, 1 in. round and carrying a 1- by 6-in. hexagonal sleeve nut, was utilized to secure the forms.

13. Forms were built in panels of various dimensions constructed of 2-in. lagging with 4- by 4-in. studding spaced 14 in. on centers with 2- by 6-in. walers spaced 3 ft 4 in. apart (Figure 2(b)). Sidewall armor was bolted to the forms before erection and anchored in the new concrete with steel straps. The take-up, necessitated by the curvature of the wall armor, was obtained by using 1/2-in. plywood.

14. When completed, the forms were filled with graded crushed limestone aggregate placed in layers, using a 1/4-cu yd crawler crane. Pyramiding was avoided in order to prevent segregation and maintain proper grading. The limestone was tamped and rodded during placing to keep voids to a minimum. Specified aggregate gradings were as follows:

Fine Aggregate	
Sieve Designation U.S. Standard Square Mesh	Percent Passing By Weight
No. 16	95-100
No. 30	60- 85
No. 50	20- 45
No. 100	15- 30
No. 200	0- 5

Coarse Aggregate		
Sieve Designation U.S. Standard Square Mesh	Percent Passing By Weight	
	Aggregate size No. 4 to 3/4 in.	Aggregate size 3/4 to 1-1/2 in.
1-1/2 in.	. . . .	90-100
1 in.	. . . .	20- 45
3/4 in.	90-100	0- 10
3/8 in.	30- 55	0- 5
No. 4	0- 5	

15. Each mortar batch consisted of the following components: three bags (94 lb each) of air-entraining portland cement conforming to Federal Specification SS-C-192, Type 1-A; one bag (75 lb) of Alfesil, a pozzolanic material supplied by the Concrete Chemicals Company, having a specific surface of not less than 3,000 sq cm per gram, and composed essentially of compounds of silicon, aluminum, and iron which combine with the lime liberated during the hydration of portland cement; 4 cu ft of sand of which from 1 to 3 percent passed the U.S. Standard No. 200 square mesh screen; 2-1/2 lb of Intrusion Aid, also manufactured by the Concrete Chemicals Company, which imparts to the mortar properties of colloidal suspension; and 18 gal of water.

16. The mortar was thoroughly mixed for at least 1-1/4 min, or until a smooth slurry of about the consistency of thick cream was obtained. The mixture was kept agitated to ensure its uniformity until it was pumped into the forms. The mortar was designed so that no appreciable set would occur for 1-1/2 to 2 hr after mixing was started so as to avoid seams or horizontal joints in it due to the layer pumping procedure.

17. The mortar mixture was fed by gravity to a three-cylinder, air-operated pump from which it moved through three 1-in. rubber hose lines attached to 3/4-in. pipes approximately 2 ft in length which were inserted through the forms. The pipes, each provided with a valve at the connection to the hose, were placed initially 4 to 5 ft apart along a horizontal line at the lowest elevation of the space to be filled.

18. Each cylinder of the pump was equipped with a bypass which permitted closure of any of the valves leading to the hose lines in case of clogging of the lines or when the discharge pipes were moved to other

locations in the form. Pumping was continued until the mortar appeared in holes provided in the form about 18 in. above the pipe centers. The pipes were then removed, the pipe entry points plugged, and the pipes raised to the next line of holes.

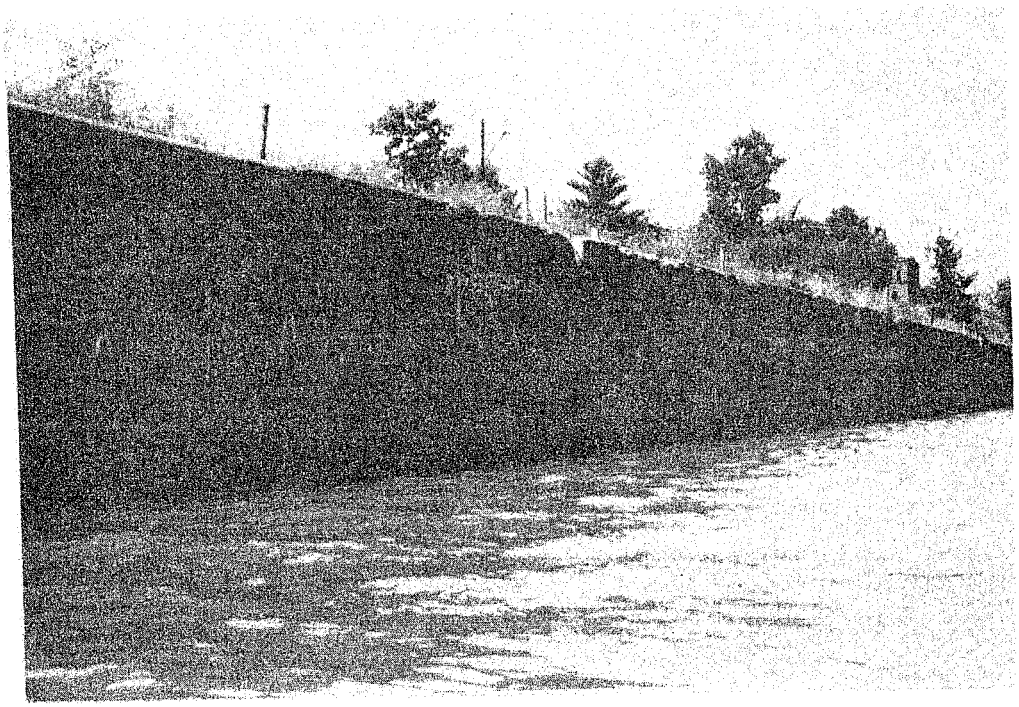
19. This operation continued until the grout appeared at the top of the form, indicating that the entire mass had been consolidated. The top surface was given a wood-float finish. Forms were left in place at least 48 hr after pumping was completed, and the concrete was cured with water for 14 days.

20. There were, of course, no horizontal joints within the new concrete since the refacing of each monolith was completed without interruption. Vertical joints, which coincided with the joints of the original monolith, were treated in a conventional manner by bulkheading. Expansion joints were made by installing premolded asphaltic joint filler after both surfaces of the joint had been painted with bituminous material.

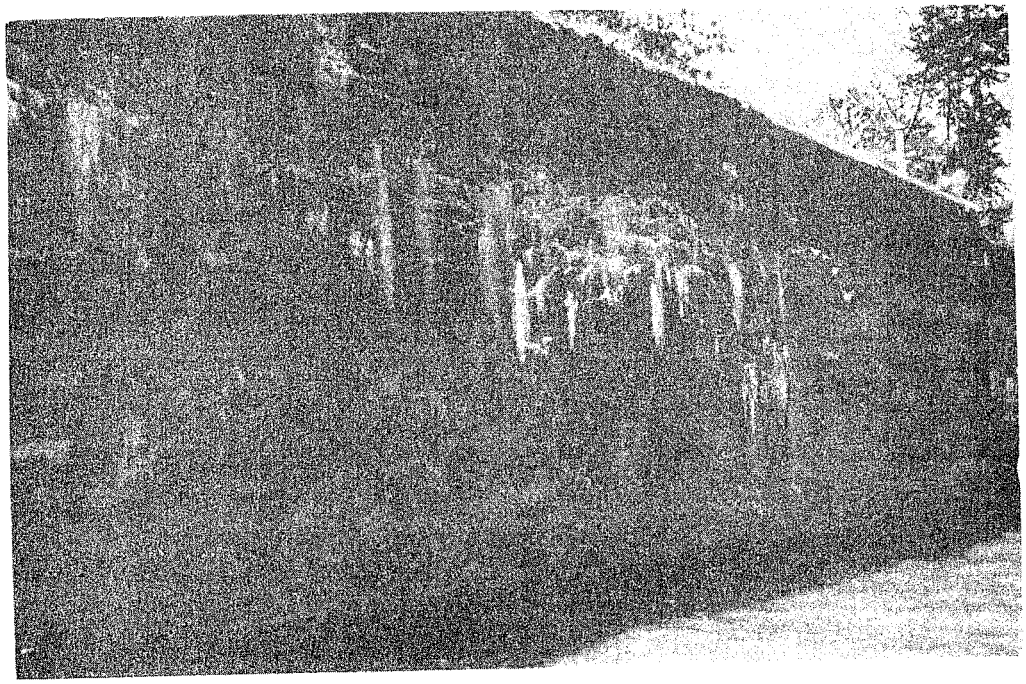
21. Equipment used, in addition to that for mixing and pumping the mortar, included two 100-ft barges, two wagon drills, and one 1/4-cu yd crawler crane. The project force averaged 55 men, working one shift the greater part of the time. All the contract work, including cleaning up, was completed in five months.

22. Contract specifications required that concrete have a minimum compressive strength of 3,500 psi at 28 days. Steel molds were used for the forming of test cylinders. The cylinders were made by filling the molds with coarse aggregate and pumping the mortar mix into the aggregate through an insert in the base of the molds. The 6- by 12-in. test cylinders showed compressive strengths ranging from 2,880 to 4,300 psi. The average strength of seven cylinders was 3,800 psi. Results of compressive-strength tests on 6- by 12-in. cored cylinders, drilled from the refacing concrete one year later, ranged from 4,060 to 7,000 psi, the average of four cores being 5,385 psi.

23. Lock No. 5 was removed from service and, with the exception of the land wall, razed in conjunction with the construction of Maxwell Lock and Dam in 1964. A visual examination of the remaining wall in July 1985 showed that the preplaced-aggregate concrete had some cracking and leaching (Figure 3) but overall appeared to be in generally good condition after 35 years exposure.



a. Overall view



b. Close-up

Figure 3. Condition of land wall, July 1985, Lock No. 5, Monongahela River

Marseilles Lock

24. The lock is located at the downstream end of the Marseilles Canal at mile 247 on the Illinois Waterway (Figure 4). The lock chamber is 110 ft wide and 600 ft long with a normal lift of 21 ft. The lock has concrete gravity walls and miter gates. The project was completed in 1933.

25. The lock was dewatered for major repairs during January and February 1952. The repair work included the following major items: (a) repair upper and lower gates, (b) reconstruct upper and lower miter sills, (c) repair recess in guide walls for cofferdam, (d) sandblast and paint upper and lower gates, (e) valve and valve well repairs, (f) reconstruct stop log recesses (valve), and (g) install flange reinforcement ties in upper and lower gates. Total cost of the repairs was $331,500. Although there appeared to be some slight scaling of the lock chamber walls between upper and lower pool elevations (Figure 5), overall the concrete appeared to be in good condition.

26. After almost 30 years exposure to severe weathering, horizontal concrete surfaces had scaled to an average depth of 3/4 in. Similar conditions existed on vertical surfaces, particularly at the joints. In addition, abrasion by vessels and ice had chipped concrete surfaces, particularly at upper and lower pool elevations and at the top of the wall (Figure 6). As a result, it was proposed to resurface portions of the top surface of the lock walls (Chicago District 1963). Also, portions of the vertical surfaces near the top of the walls were to be resurfaced and corner armor installed (Figure 7). This work was completed in 1965 at a cost of $80,000.

27. The first periodic inspection of Marseilles Lock was conducted in August 1967. Concrete erosion of the chamber walls above low pool was reported to be in excess of 6 in. deep in some areas. The deterioration was attributed primarily to cycles of freezing and thawing of the wet concrete surfaces and abrasion by barges during normal operation of the lock. Tops of the walls resurfaced in 1965 remained in good condition.

28. The second periodic inspection was conducted in August 1972. Concrete resurfacing on top of the north lock chamber wall was in good condition; however vertical surfaces were severely deteriorated, particularly at monolith joints (Figure 8). Similar conditions existed on the south wall with the addition of a few minor cracks on top of the wall. The chamber walls had eroded to the point that the downstream miter gate was vulnerable to barge

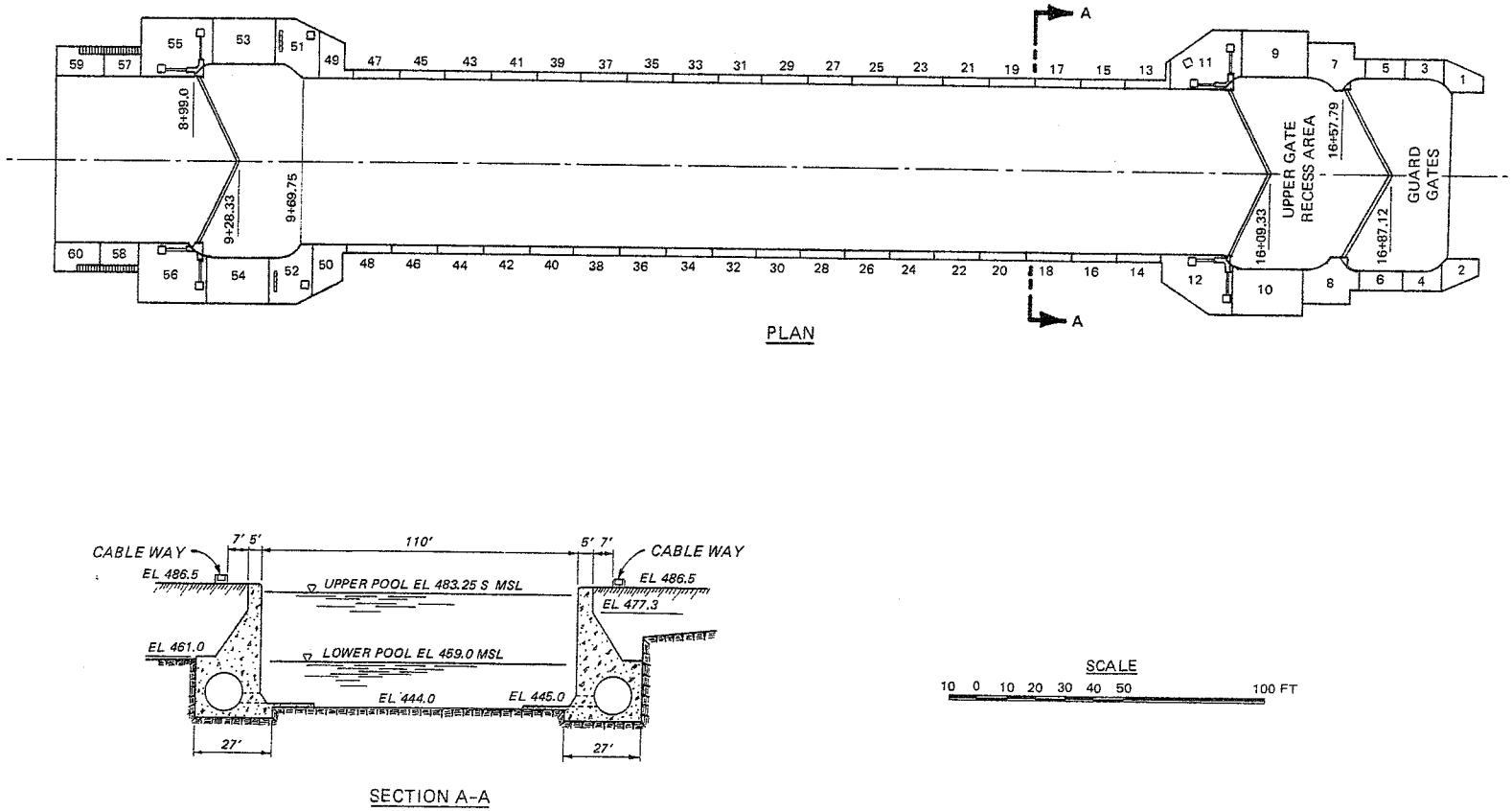


Figure 4. General plan and elevation, Marseilles Lock

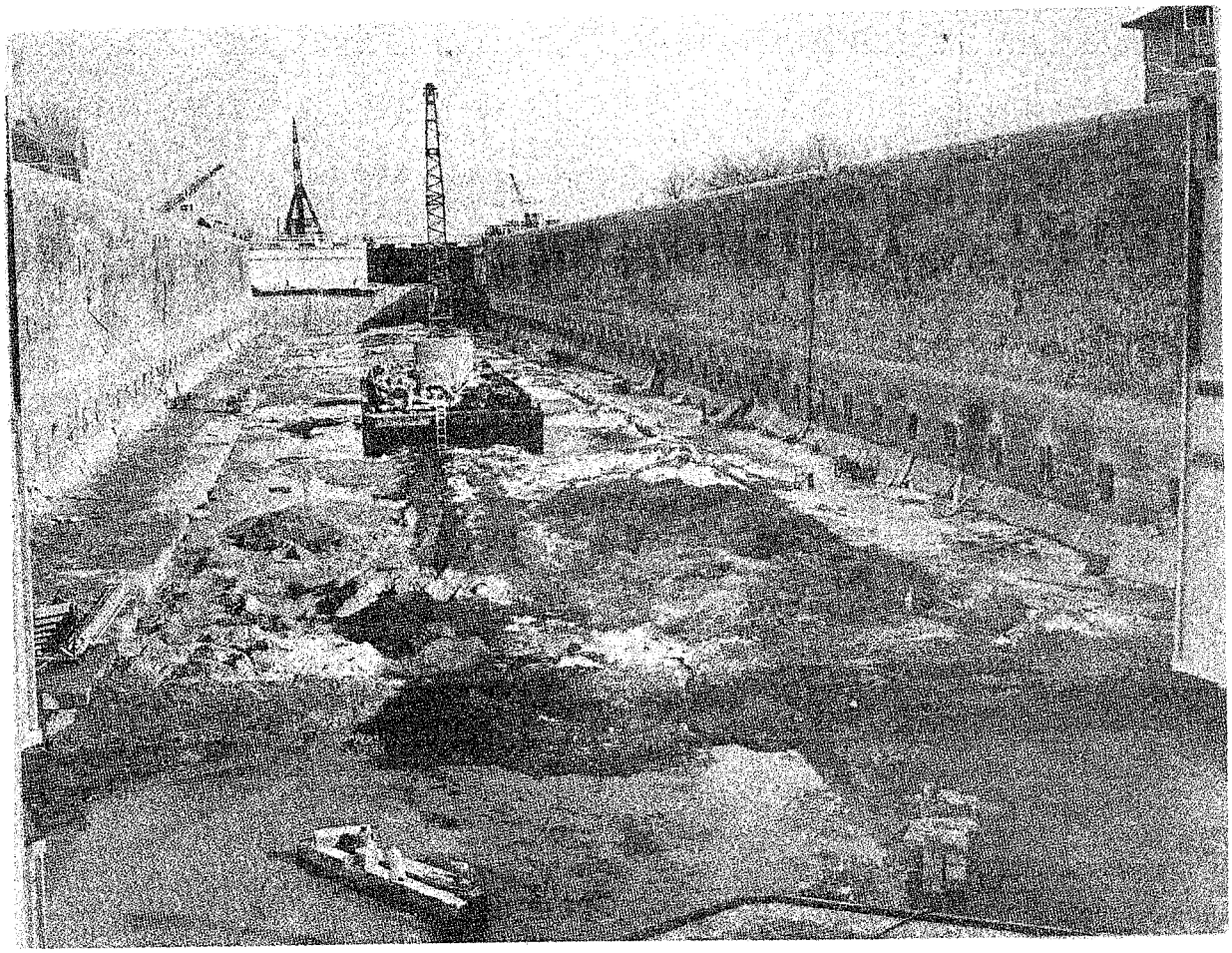
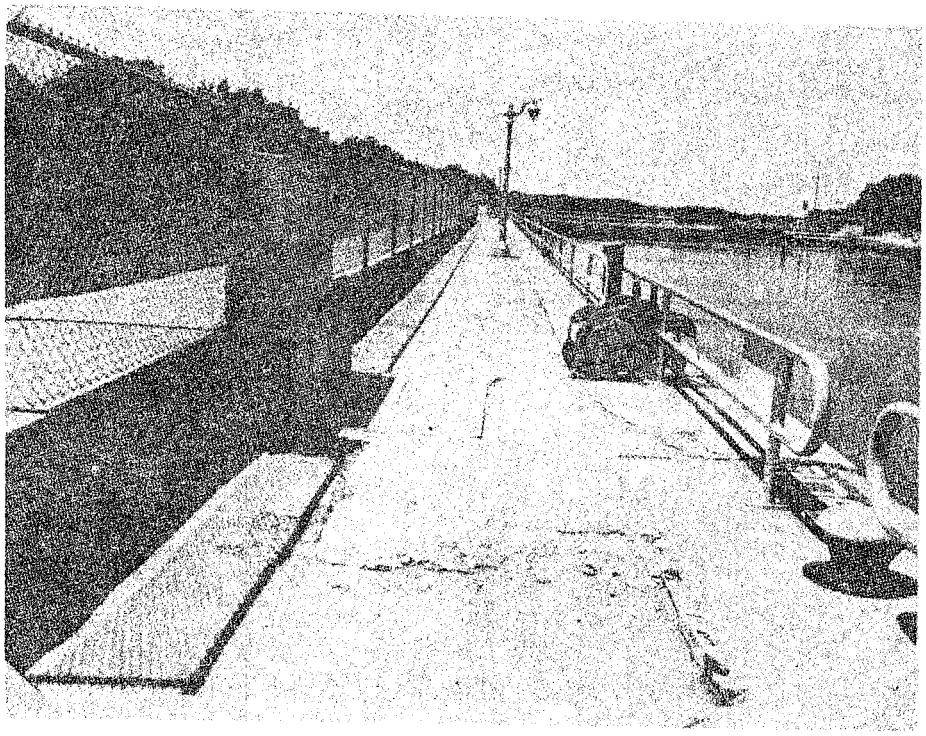


Figure 5. Lock chamber dewatered for repairs, 1952, Marseilles Lock

traffic when the gate was in the open position. It was concluded that if allowed to progress, loss of section through erosion would impair the stability and integrity of the lock walls. Therefore, it was recommended that the lock chamber walls be resurfaced. Since such a repair would have to be accomplished with a minimum of lock downtime, the Office of Chief of Engineers (OCE) suggested that precast concrete stay-in-place forms be studied as a potential repair technique. The Little Rock District (1963) had investigated this system of forming for new construction, and the Bureau of Reclamation had used it in the restoration of Barker Dam (Davis 1948).

29. In 1973, the depth of concrete erosion was established through a detailed survey of the vertical surfaces of the lock chamber walls. In the survey, erosion depths were measured at 5-ft intervals horizontally and vertically. Generally, the depth of erosion ranged from 0.20 to 0.65 ft, with an overall average of about 0.40 ft of concrete eroded from the original surface.



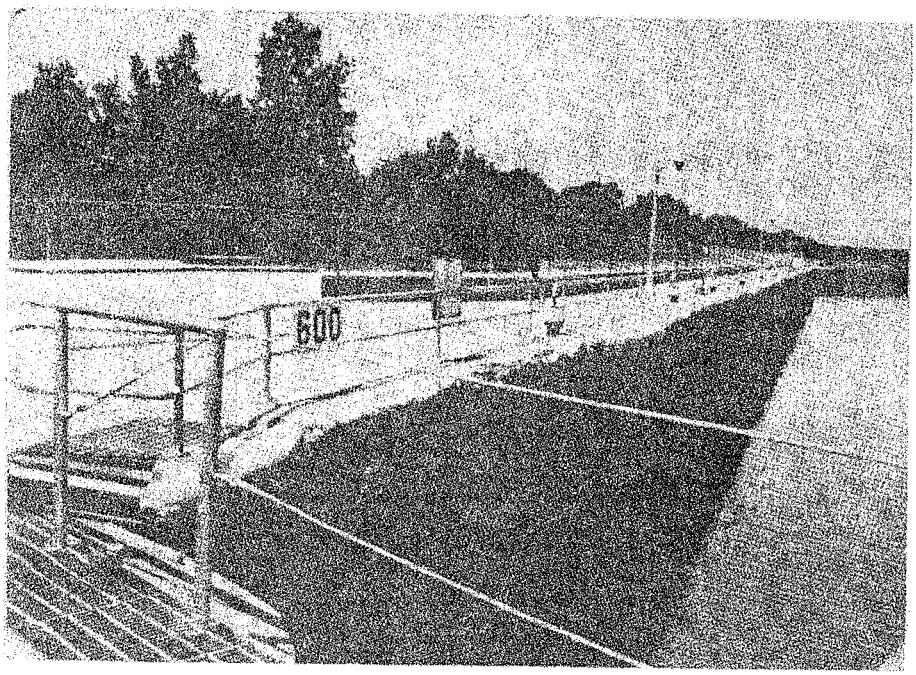


Figure 6. Typical concrete deterioration, 1962, Marseilles Lock

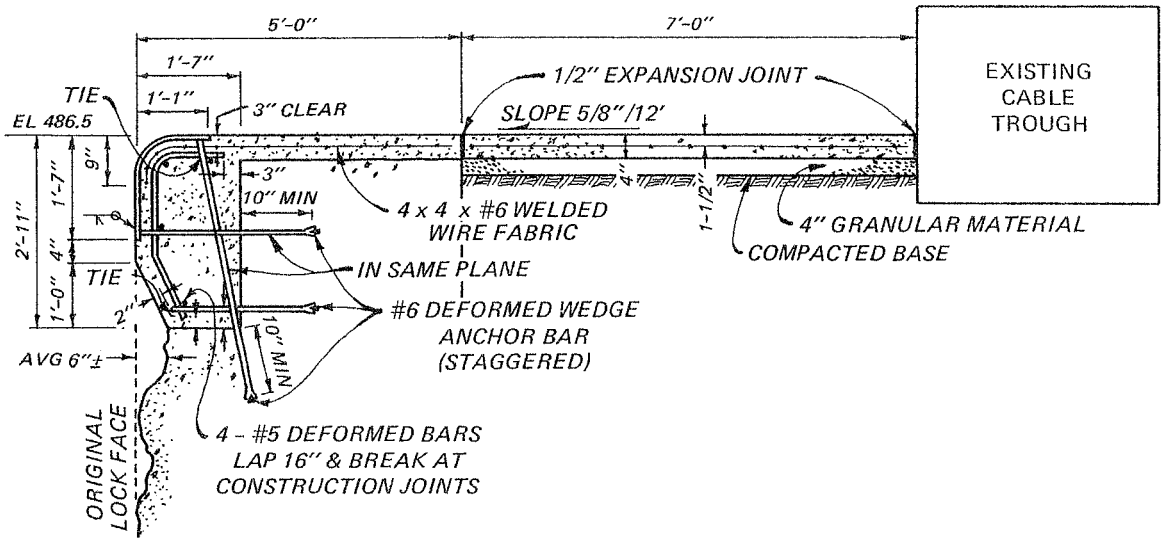


Figure 7. Typical resurfacing and corner armor installation details, 1965, Marseilles Lock

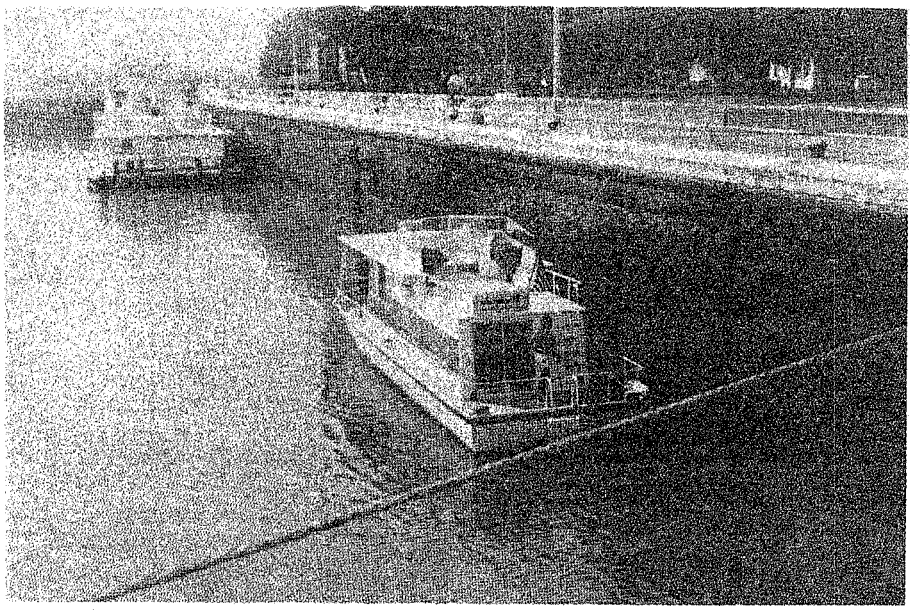


Figure 8. Severe deterioration of north lock wall, 1972, Marseilles Lock

The maximum depth of erosion (1.10 ft) was measured in a few isolated areas. The surface of a typical monolith (31) is shown in Figure 9, and the results of the survey on the monolith are shown in Figure 10. Erosion of the concrete ranged from slightly above low pool elevation to approximately 2 ft below the top of the wall with the most severe erosion in the upper portion of the walls.

30. A program to assess the quality of the remaining concrete and to establish the depth of deteriorated concrete was also conducted. Six horizontal concrete cores, 6-in. diameter by about 3 ft long, were obtained from each wall and tested at the Ohio River Division Laboratory (Chicago District 1974). A typical log of the concrete core is shown in Figure 11. Specimens for compressive strength tests were taken from those portions of the cores located a minimum of 0.7 ft behind the existing face of the wall. Compressive strengths ranged from 5,070 to 7,760 psi with the average of nine tests being 6,440 psi. No increase in strength was noted with increasing depth; in fact, the deeper test specimens generally exhibited lower compressive strengths. Depth of fracturing in the concrete, as determined by petrographic examination, ranged from 0.1 to 1.1 ft with an average depth of 0.5 ft. Other observations based on the petrographic examination are summarized in the following:

a. Although the coarse aggregate was predominantly dolomite and dolomitic limestone, no evidence of carbonate-alkali reactivity was noted.

b. Aggregate did not appear "dirty," but several particles with weathered rims were noted in each boring.

c. Most of the weathered particles appeared to be weathered siltstones which were quite soft, porous and absorbant, and susceptible to freeze-thaw action.

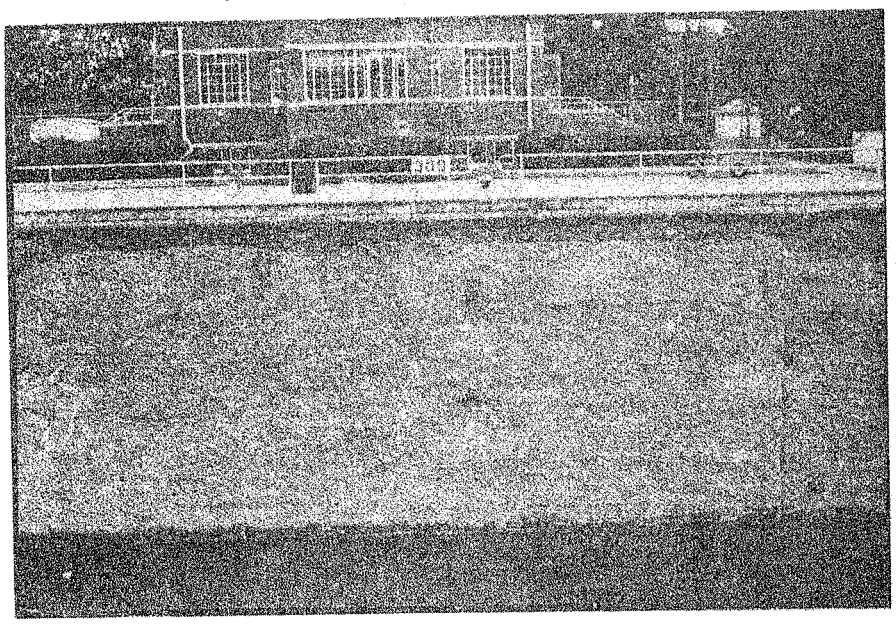
d. The depth of iron-staining appeared to be the depth of penetration of water from the lock chamber. This surface was very irregular and always deeper than the depth of deterioration of the concrete.

e. The series of fractures parallel to the lock wall surface which were present in the top portion of each boring appeared to be due to freeze-thaw action.

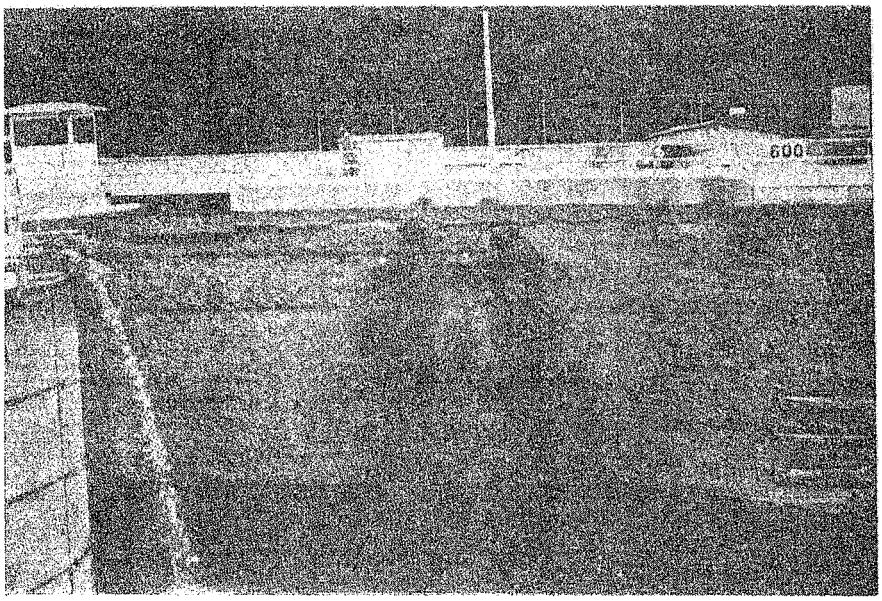
f. The percent of air voids in the specimens ranged from 0.72 to 2.12.

g. The air voids were due to entrapped air and were generally associated with the porous dolomite particles.

h. The concrete contained no entrained air.



a. Monolith 31



b. Gate block monolith

Figure 9. Typical concrete conditions, 1973, Marseilles Lock

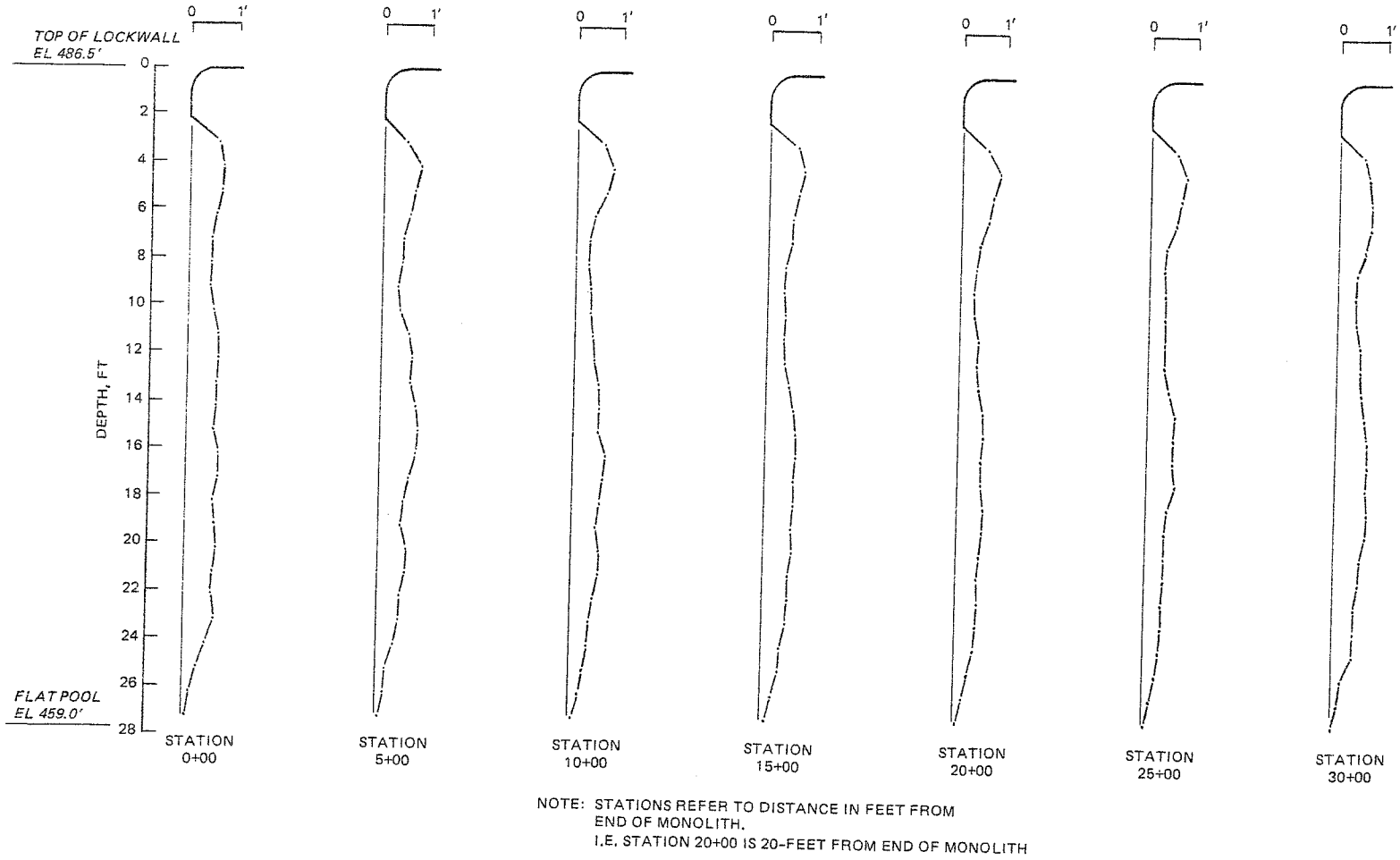


Figure 10. Cross section of monolith 31, 1973, Marseilles Lock

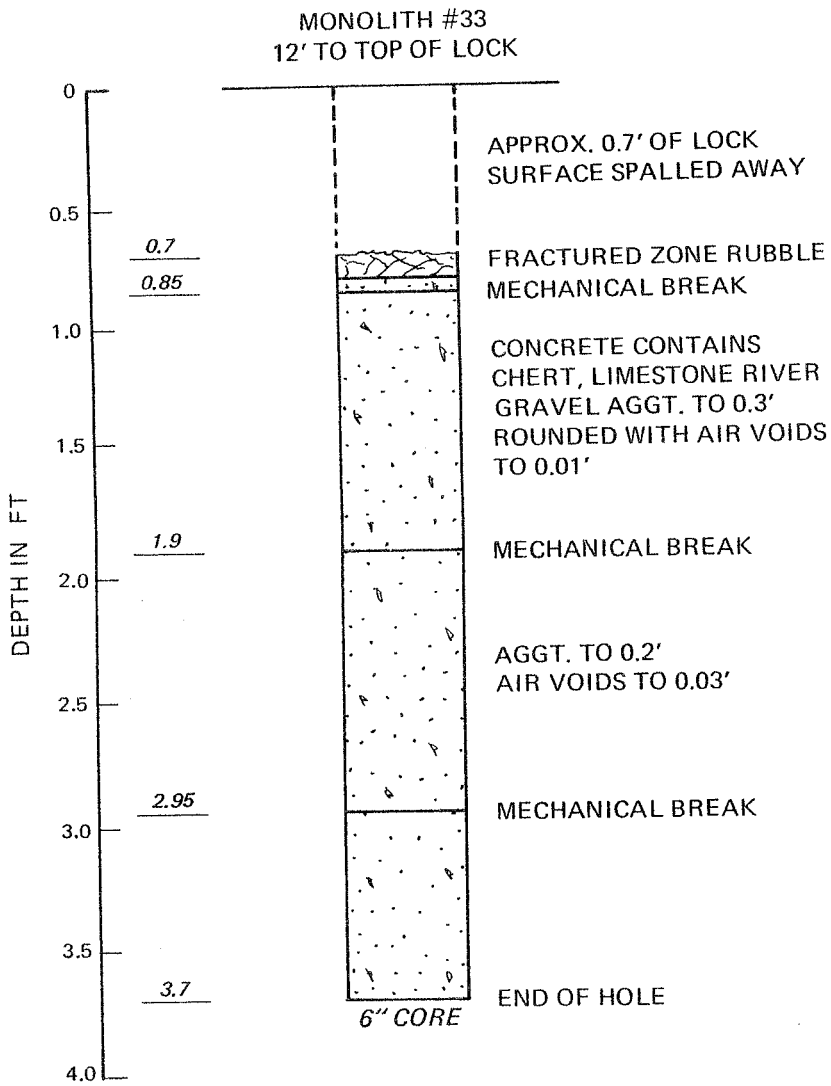


Figure 11. Typical concrete core log, 1974, Marseilles Lock

i. The depth of deterioration appeared to be greater between elevations 482.7 and 471.5 (average of 1.08 ft) than between elevations 471.5 and 465.0 (average of 0.76 ft).

j. The core from all borings was tested with the Schmidt Impact Hammer. The results were inconclusive due to the inconsistency and variability of the readings.

No conclusions were drawn from the test results regarding the causes of concrete erosion. Lack of air entrainment, weathering of siltstone particles, and severe freeze-thaw environment were believed to be some of the factors contributing to the erosion.

31. Since it appeared that erosion had progressed to the point that the integrity of the structure was endangered, a study of materials and

construction methods for resurfacing the lock chamber walls was conducted (Chicago District 1974). Various methods used for refacing the hydraulic structure were studied including shotcrete, preplaced aggregate concrete, conventionally placed concrete, precast concrete panels, and epoxy mortar. Case studies were made of previous resurfacings to evaluate methods as to their performance and suitability for lock resurfacing. In addition, cost and time estimates were made for each acceptable scheme under various operational requirements of the lock during construction.

32. Only those methods and procedures which had been successfully used for similar types of resurfacing were considered. Each method was selected on the basis of the following criteria:

a. It must be thoroughly and permanently bonded to the existing concrete.

b. It must be sufficiently impermeable to prevent moisture reaching underlying existing concrete.

c. It must, after drying, be free of shrinkage cracks through which water could reach the supporting concrete.

d. It must be resistant to cyclic freezing and thawing.

e. It must have sufficient resistance to abrasion.

f. It must not require a long time to complete.

33. Based on an evaluation of the advantages and limitations of the various repair methods, detailed cost estimates were prepared for the following:

a. Shotcrete. Pneumatic application of concrete to lock surface by either dry or wet process.

b. Precast Panels with Cast-in-Place Concrete. The method consists of anchoring 4-in.-thick precast panels and placing concrete in the space between the panels and the existing lock wall by conventional methods (Figure 12).

c. Precast Panels and Preplaced Aggregate Concrete. Same as b above, except the space between the panels and the lock walls is filled by preplaced aggregate concrete.

d. Conventional Concrete. In this method, forms are erected and concrete is placed by the tremie method.

e. Preplaced Aggregate Concrete. In this method, the space between the forms and the lock walls is filled with preplaced aggregate concrete.

In all cases, removal of existing concrete to sound concrete is required. For cost comparison purposes it was assumed that, on the average, a 2-in. layer of surface concrete would have to be removed.

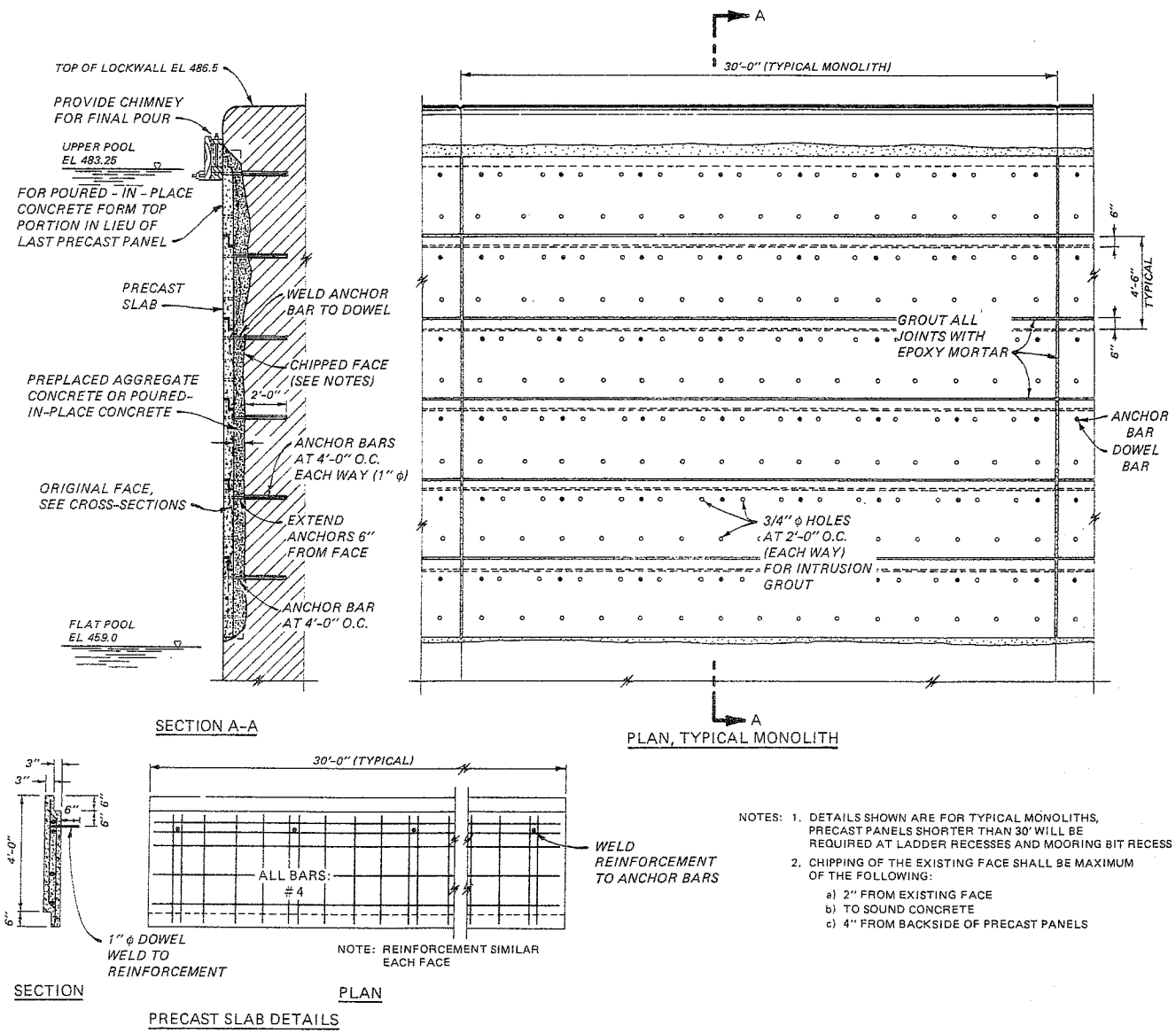


Figure 12. Lock wall resurfacing using precast panels, Marseilles Lock

34. Three construction schedules were studied for each repair method.

a. Schedule A. Close the lock to navigation and perform all phases of construction in the dry.

b. Schedule B. Perform removal of concrete to sound concrete by closing of the lock for 8 hr/day. Close the lock upon completion of removal and complete remainder of construction in the dry.

c. Schedule C. Under this schedule, complete closing of the lock was not required. However, for part of the construction period, the lock operations were restricted to 8 hr/day for the shotcrete repair and 16 hr/day for the other repairs. Also, during the entire construction period, lockages were limited to two barge widths.

35. Estimates of cost and construction time for the various repair procedures are summarized in the following:

Repair Procedure	Construction Cost, $K	Construction Time, Work Days: Lock Closed 8 hr/day*	Construction Time, Work Days: Lock Closed 24 hr/day**	Construction Time, Work Days: Total Construction Time†
	Schedule A			
Shotcrete	672		21	30
Precast panels and cast-in-place concrete	1,786		30	40
Precast panels and preplaced aggregate concrete	1,842		30	40
Conventional concrete	726		26	36
Preplaced aggregate concrete	732		26	36
	Schedule B			
Shotcrete	624	12	20	48
Precast panels and cast-in-place concrete	1,687	25	25	70
Precast panels and preplaced aggregate concrete	1,742	25	25	70

(Continued)

* Five-day week.
** Seven-day week.
† Calendar days.

Repair Procedure	Construction Cost, $K	Construction Time, Work Days: Lock Closed 8 hr/day	Lock Closed 24 hr/day	Total Construction Time
	Schedule B (Continued)			
Conventional concrete	678	12	24	54
Preplaced aggregate concrete	684	12	24	54
	Schedule C			
Shotcrete	762	60		84
Precast panels and cast-in-place concrete	1,372	60		84
Precast panels and preplaced aggregate concrete	1,416	60		84
Conventional concrete	638	60		84
Preplaced aggregate concrete	643	60		84

36. The cost comparisons indicate that shotcrete and formed concrete, either conventional or preplaced aggregate concrete, are comparable in cost. Shotcrete was more economical under construction Schedules A and B while formed concrete was more economical under Schedule C. Removal of concrete by closing of the lock for 8 hr each day and complete closing of the lock for shotcreting operations was found to be the most economical. However, it was the opinion of the Operations Division that the impact on navigation would be less if the lock were completely closed for the entire construction, and this reduction would more than offset the added cost of construction. Also, there were some apprehensions concerning the removal of concrete with the lock in operation. It was feared that accidental dropping of concrete debris during removal could clog the lock valves.

37. Shotcrete was found to be slightly less costly than the selected repair plan using formed concrete. Shotcrete was not selected because it was generally agreed that the quality of shotcrete is dependent to a large degree on the skill of the nozzleman. Therefore, rigid and extensive controls would be required for quality assurance of shotcrete repairs. Also, should removal of unsound concrete be more than assumed, the cost of shotcrete would rise

sharply, while it should remain relatively the same for the formed concrete. Therefore, it was proposed to chip back the wall surfaces to sound concrete, install reinforcement, place form work to the original surface line of the lock, and place either conventional concrete or preplaced aggregate concrete.

38. The average depth of concrete removal was estimated at 2 in.; however removal to sound concrete would be established by judgment. Absence of visible cracks and absence of a flat, dull sound as the concrete surface is hit by a hammer would be the basic criteria for establishment of sound concrete. It should be noted that it was not proposed to remove the surface concrete to the depth of microscopic cracks, average depth of microscopic cracks being 6 in. It was believed that anchored refacing made removal to the depth of microscopic cracking unnecessary and also the depth of microscopic cracking could not be established in the field.

39. Construction was to be done with the lock completely closed and work performed 24 hr/day using three work shifts each day. It was estimated that the construction with the lock closed would require 26 calendar days to complete on the basis of a 7-day work week. The total construction time including mobilization and demobilization would be 36 calendar days.

40. Resurfacing of the lock chamber walls was accomplished during a 30-day period in April and May 1975. Specifications required removal of existing concrete to sound concrete and limited the methods of removal to hand-operated tools. Blasting was not permitted to avoid cracking of the remaining concrete. After award of the contract, the contractor submitted a Value Engineering (VE) proposal, to remove the concrete by close drilling and blasting. In evaluation of the VE proposal, the contractor was allowed to test-blast a section of the lock. The purpose of the test blasting was to ensure that no cracks developed in the lock walls and also to determine the optimum blast-hole spacing for removal of concrete. Spacings of 8, 10 and 12 in. were tried. Based on results of the test, a spacing of 10 in. was adopted for production removal. Concrete was removed to a depth of 14 in. from the original face of the wall.

41. Corner armor previously installed was left in place during resurfacing of the chamber walls. Blast holes and holes for concrete placement by pumping were drilled behind the existing armor (Figure 13). The replacement concrete was placed to the underside of the resulting overhang. The cavity between the new concrete and the existing concrete was filled with epoxy

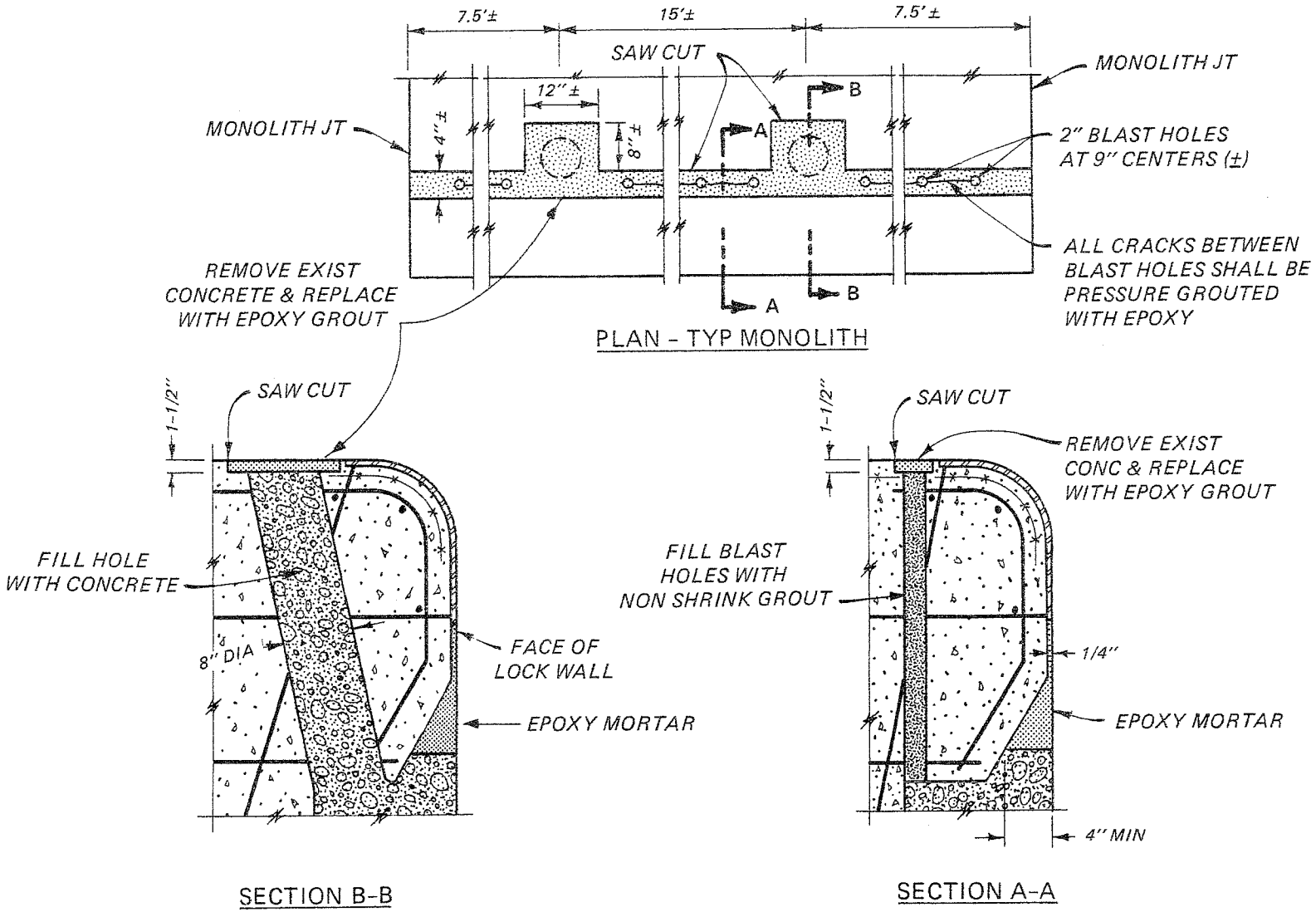


Figure 13. Resurfacing details, Marseilles Lock

mortar. Tremie holes and blast holes were filled with nonshrink grout and were topped with a layer of epoxy mortar. The cracks that resulted from the blasting operations were pressure grouted with epoxy. Below the top of the wall, resurfacing details were the same as for Dresden Island.

42. The epoxy applications were a "fiasco" (Juzenas 1979). Delamination of the epoxy layer below the top corner protection plate of the lock wall started within days after reopening of the lock to the waterway traffic. In many places the epoxy mortar was rubbery and easily bent by hand. The Waterways Experiment Station (WES) was engaged to determine the causes of the failure. Upon performance of a battery of tests on the materials and on samples taken from the wall, WES concluded that delaminations were predominantly caused by barge impacts on the epoxy mortar overlay that was poorly bonded to the underlying material. Lack of cleaning and smoothness of the underlying material were thought to be the principal contributing factors in producing a poor bond. The extraction tests indicated that insufficient curing caused the epoxy mortar to be soft. Insufficient curing was attributed to improper mixing or proportioning. As a result of the epoxy mortar failure, a horizontal steel plate was used to cover the failure area (Figure 14).

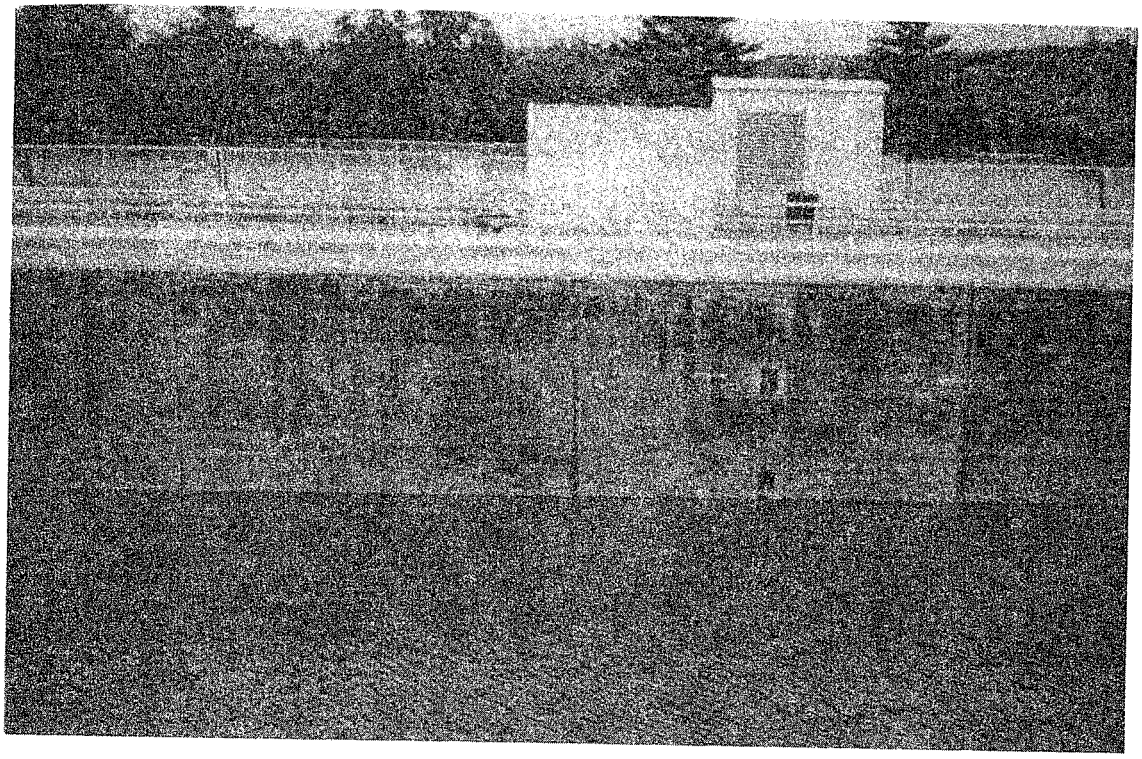


Figure 14. Steel plate below corner protection armor used to cover expoxy mortar failure, May 1985, Marseilles Lock

43. The epoxy layer at the top of the lock also delaminated (Figure 15). A large percentage of the epoxy overlay has spalled off, is cracked, or has disintegrated. A large thermal coefficient of expansion of the epoxy mortar as compared to that of the concrete is the most likely cause of failure.

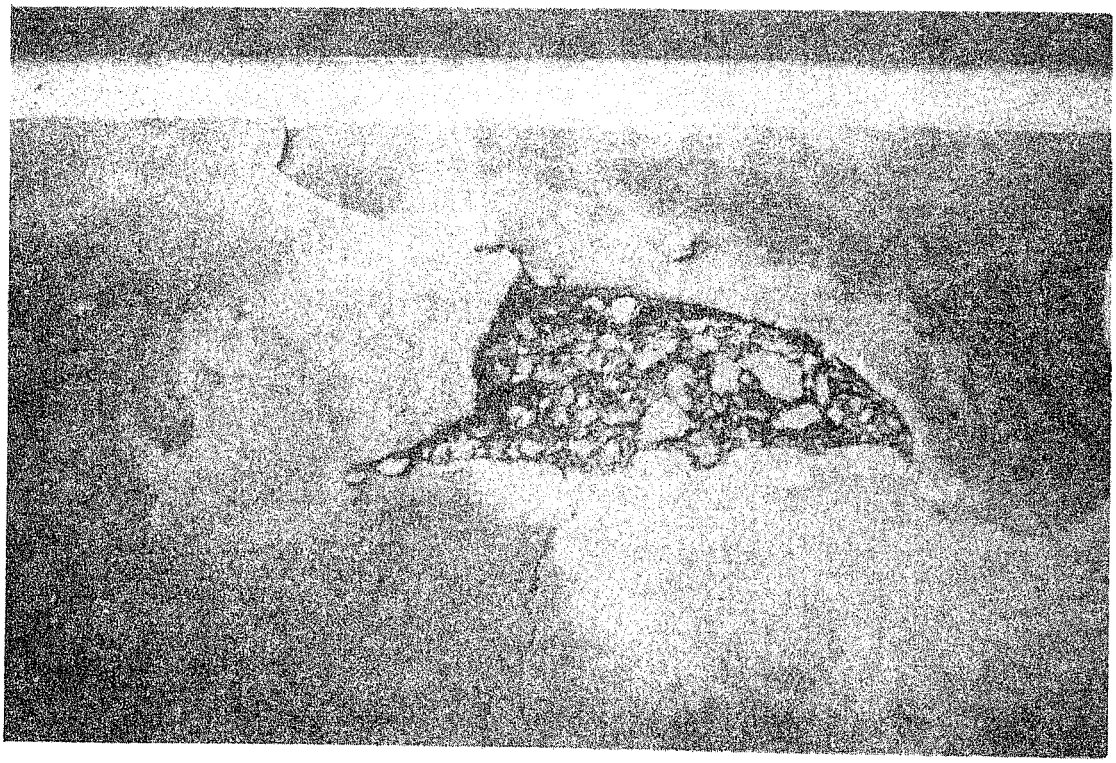
44. The condition of the replacement concrete was examined from a workboat during an emptying cycle as part of the third periodic inspection in August 1977. A pattern of tight vertical cracks at about 4- to 5-ft spacing down to low pool was reported on the right (north) wall. A similar pattern was not observed on the south face; however deposits and staining made detection of any fine cracks very difficult. The cracks observed were not considered uncommon and were attributed to volume changes in the concrete related to drying, temperature, or both. It was concluded that with the exception of the minor cracking noted, the resurfaced walls appeared to be in generally good condition. This minor cracking, which was observed upon completion of resurfacing in 1975, had not resulted in any apparent deterioration at the replacement concrete. It was recommended that resurfacing of the gate bays, which was not accomplished during the 1975 contract because of time limitations, be completed.

45. A contract was advertised in April 1978 for removal of surface concrete from the gate bays and the downstream face of the sill of the upper service gate and replacement with new concrete, including grouted anchors and metal wall-armor tees; removal of existing corner protection plates and installation of new ones in the gate bays, forebays, and the upper guide wall; and rehabilitation of the miter gate machinery. The low bid was submitted by Thomas Madden Company, Chicago, Illinois, in the total contract amount of $1,265,000. The majority of this work was completed during a two-month shutdown of the lock starting 1 August 1978.

46. The fourth periodic inspection of Marseilles Lock and Dam was conducted in September 1982. The condition of the concrete in the lock was described as follows:

a. Upper right guide wall: The surface of the guide wall is spalled in places, especially around lamp pedestals, embedded steel plates, and joints. The river edge of the wall has been resurfaced and is generally still in good condition. Alignment of the guide wall is straight.

b. Lock:



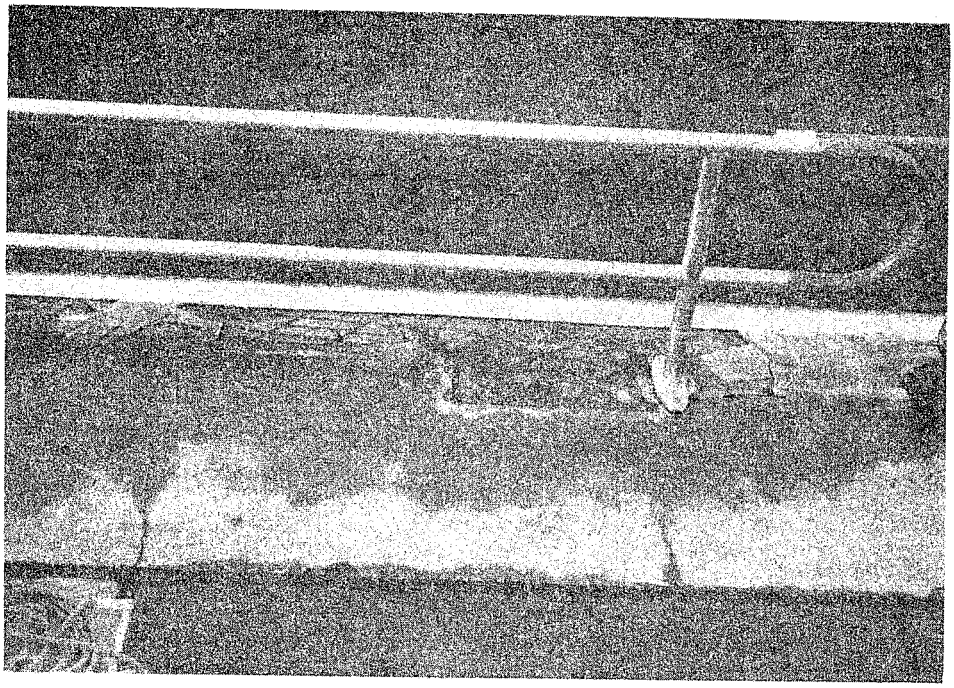


Figure 15. Typical delamination of epoxy overlay, May 1985, Marseilles Lock

(1) The top surface of the concrete in the upper right emergency gate bay monolith is moderately scaled. The upstream right gate monolith has reflective cracks that are typical of all resurfaced gate monoliths. There are weeds growing out of the monolith joint next to the downstream right service gate.

(2) The sidewalk adjacent to the right lock wall monoliths has subsided. According to the lockmaster, Walt Gatza, the problem began with dewatering of the lock chamber for rehabilitation work. Mud jacking was undertaken in 1978 to raise the slabs, but the problem remains.

(3) The lock chamber walls are generally in good condition, although there is some minor cracking and occasional epoxy spalled out on the top edges.

(4) The downstream vertical faces of monoliths 55 and 56, resurfaced in 1978, have large shrinkage cracks and some leaching (Figure 16).

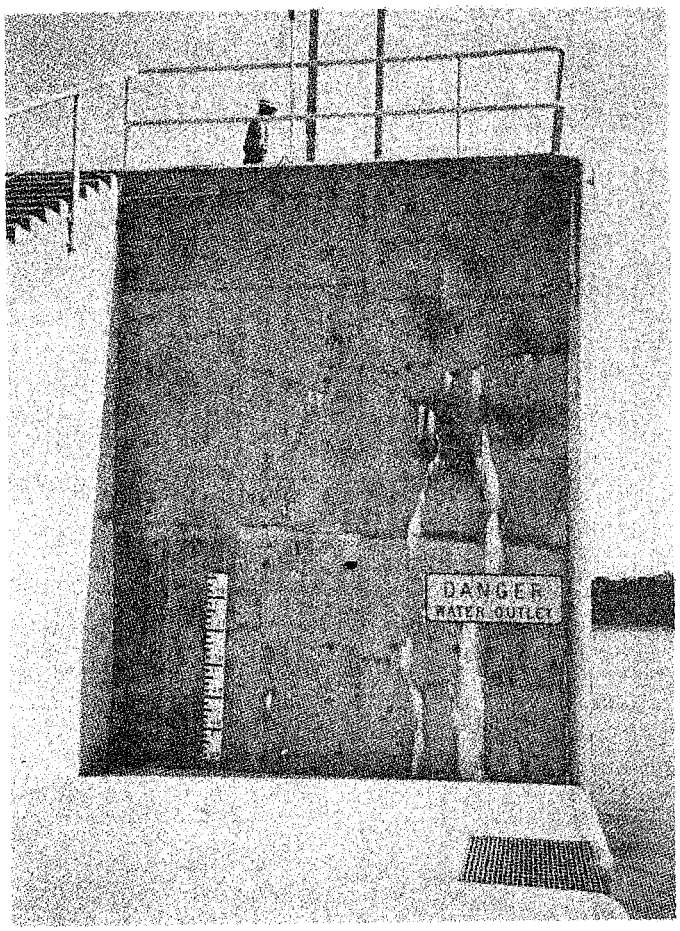


Figure 16. Cracking in downstream face of monolith 55, September 1982, Marseilles Lock

(5) The upstream end of the lower right and lower left gate bays have minor spalling along the construction joints.

(6) There is some spalled concrete on the downstream edge of the upper gate sill.

c. Lower left guide wall: The vertical face of the guide wall, especially at monolith joints, is severely weathered and suffers from abrasion due to contact with barges. The top of the wall was resurfaced in 1981 and is in very good condition, with no shrinkage cracks.

Old Lock No. 14, Mississippi River

47. The lock was constructed in 1922 in conjunction with a 2.5-mile-long canal as part of the 6-ft channel project on the Mississippi River. It is located adjacent to the new Lock and Dam No. 14 at LeClaire, Iowa, which was constructed in 1939. After construction of the larger Lock No. 14, the old lock was considered an auxiliary lock and used only for access of Corps of Engineers boats to the Rock Island District service and maintenance area. In 1969, the old lock was returned to operation for pleasure craft use on weekends and holidays from Memorial Day until the first weekend in October. Peak loads for the auxiliary lock are in excess of 400 craft carrying 1,700 to 1,800 passengers per weekend. This lock also continues to be used for access to the District service base by floating plant equipment.

48. The usable lock chamber is 80 ft wide by 320 ft long with concrete gravity walls founded on rock. The lock walls as constructed were 27 ft high measured from the lock floor, with a base width of 20 ft. Both chamber walls retain backfill material. The upper guide wall is a concrete gravity wall retaining backfill material. The downstream guide wall is a free standing, timber, rock-filled crib wall. In about 1940, the landside wall and the downstream, riverside gate monolith were modified. Six inches of the original concrete was removed and replaced with concrete containing natural gravel aggregate.

49. After 55 years exposure to the elements, lock concrete surfaces were severely scaled (Figures 17 and 18). An investigation was initiated in 1977 to determine the extent and causes of the concrete deterioration. Two 6-in.-diam holes were core drilled vertically through the landside lock wall and 5 ft into the underlying bedrock. In addition, 6-in.-diam cores were obtained from a total of 12 horizontal and vertical holes drilled to depths of

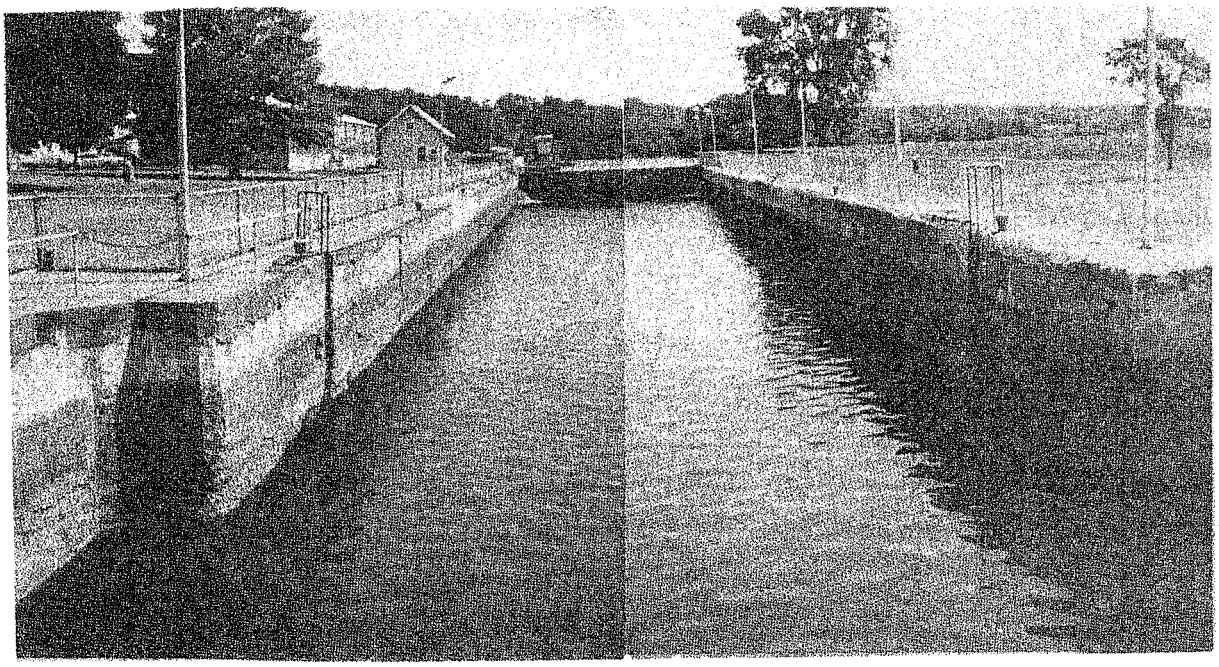


Figure 17. General view of lock looking upstream 1977, Old Lock No. 14, Mississippi River

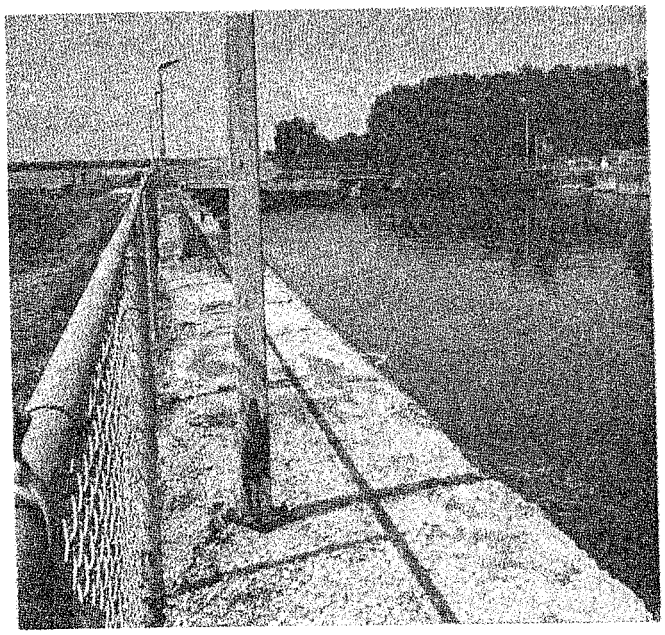
about 2 ft in each lock wall. Following a petrographic examination, compressive strength, modulus of elasticity, and resistance to freezing and thawing tests were conducted on representative cores. Results of this investigation (Rock Island District 1978) are summarized as follows:

a. The nonair-entrained concrete contained significant amounts on nondurable aggregate which had been damaged by cycles of freezing and thawing. The deterioration was considered to be progressive because the scaled areas and cracks allow more moisture to penetrate the concrete. Such moisture penetration would enhance further concrete damage when subjected to additional cycles of freezing and thawing.

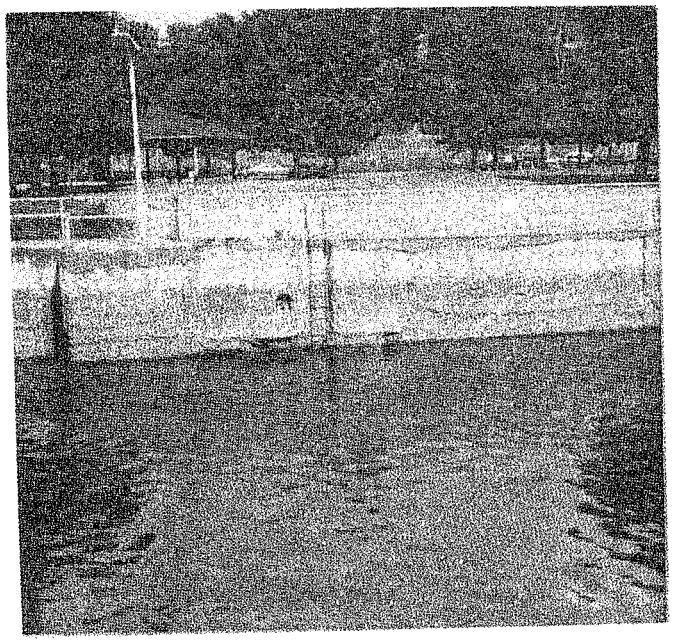
b. There were traces of silica gel and calcium sulfoaluminate in some of the cracks and air voids. However, alkali-silica reaction and sulfate attack were not considered to be of any consequence in this concrete.

c. The concrete in the top of the riverside wall was deteriorated to a depth of about 10 in. The 6-in. concrete cap on the landside wall was in good condition; however the concrete beneath the cap was deteriorated to a depth of about 12 in. The downstream gate monolith on the riverside wall exhibited a similar condition (Figure 19). The depth of deterioration on vertical surfaces ranged from 2 to 7 in.

d. The compressive strength of the concrete tested ranged from 2,000 to 3,500 psi. The average chord modulus of elasticity of the horizontal cores was 2.0 by 10^6 psi.



a. Riverside wall



b. Landside wall

Figure 18. Typical concrete deterioration, 1977, Old Lock No. 14, Mississippi River

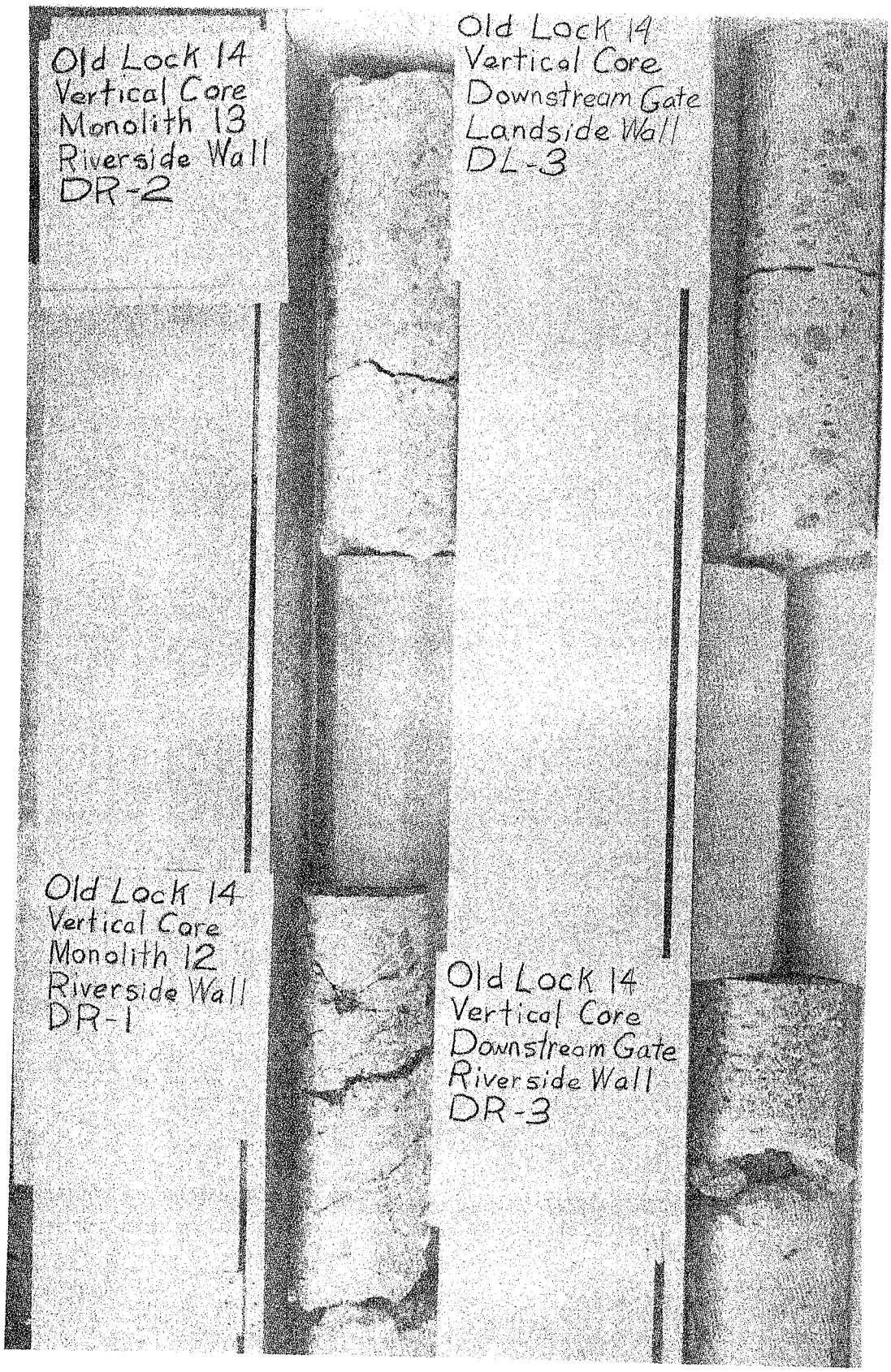


Figure 19. Photographs of selected concrete cores, 1977, Old Lock No. 14, Mississippi River

50. The rehabilitation project began with the award of a contract for the cofferdam and dewatering in July 1978. Dewatering was accomplished by constructing steel sheet pile cell cofferdams upstream and downstream of the lock and pumping with contractor-furnished pumps. Backfill excavation and dewatering were required for wall stability during the initial dewatering of the lock. The old lock floor was removed, and a new 18-in.-thick reinforced concrete slab anchored to the foundation rock was constructed. This slab increased the sliding stability of the gravity walls and eliminated the requirement for backfill excavation and dewatering for the remainder of the rehabilitation work and future dewatering. Type V cement was used in grouting floor slab anchors to provide resistance to the possibility of sulfate attack.

51. Concrete deterioration on vertical surfaces was generally confined to those areas of the lock walls above low water level (Figure 20). Therefore, concrete removal and replacement was confined to the top 17 ft of the walls except for the outer gate monoliths where extensive concrete removal was necessary to accommodate new gates and operating machinery (Rock Island District 1980). Twelve inches of concrete was removed from the vertical face of the walls, and two feet of concrete was removed from the top of the walls (Figure 21).

52. Removal of the concrete on the vertical face of the walls was accomplished by line drilling 2-1/2-in.-diam holes on 12-in. centers, loading with 50-grain detonating cord with one-fifth of a stick of water gel attached at 18-in. intervals, and stemming the entire hole. This explosive was approximately equivalent to 600-grain detonating cord. The new lock floor was protected from damage by placing timber mats on the floor in the area where debris from the blasting operation would fall. Concrete removal by line drilling and blasting proved to be efficient and precise (Figure 22). Damage to the remaining portions of the walls was minimal. Some drill holes drifted out of alignment, but most were straight and accurate.

53. The concrete on the top surface of the walls was removed by drilling 2-1/2-in.-diam holes in a grid pattern and blasting. The holes were loaded with 50-grain detonating cord with one-fifth of a stick of water gel attached at 18-in. intervals and stemmed the full depth. The debris was contained by placing timber mats on the top of the walls and hanging rubber tire mats on the vertical face of the walls. The bid prices for concrete removal was $140 per cubic yard.

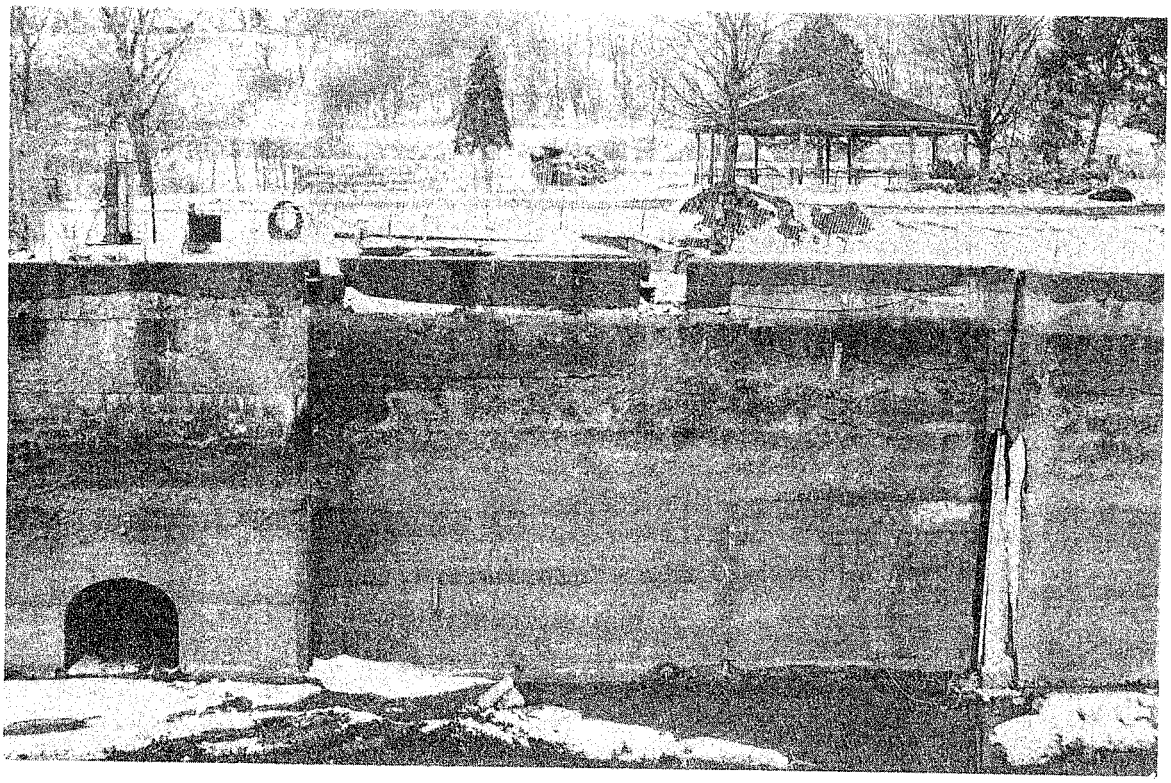


Figure 20. Typical concrete deterioration, February 1980, Old Lock No. 14, Mississippi River

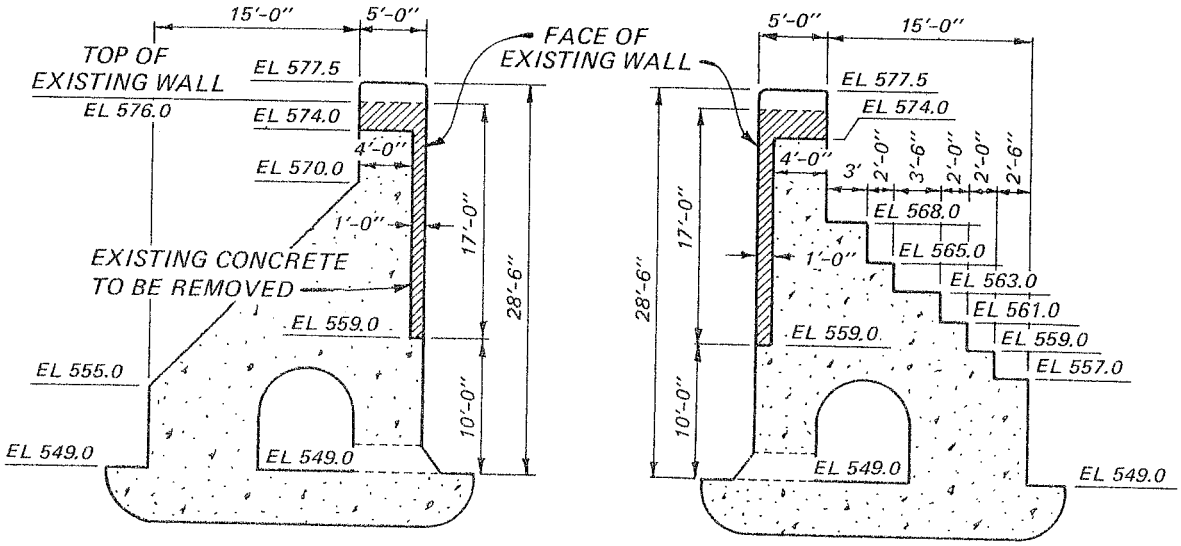


Figure 21. Typical sections showing extent of concrete removal and replacement, Old Lock No. 14, Mississippi River

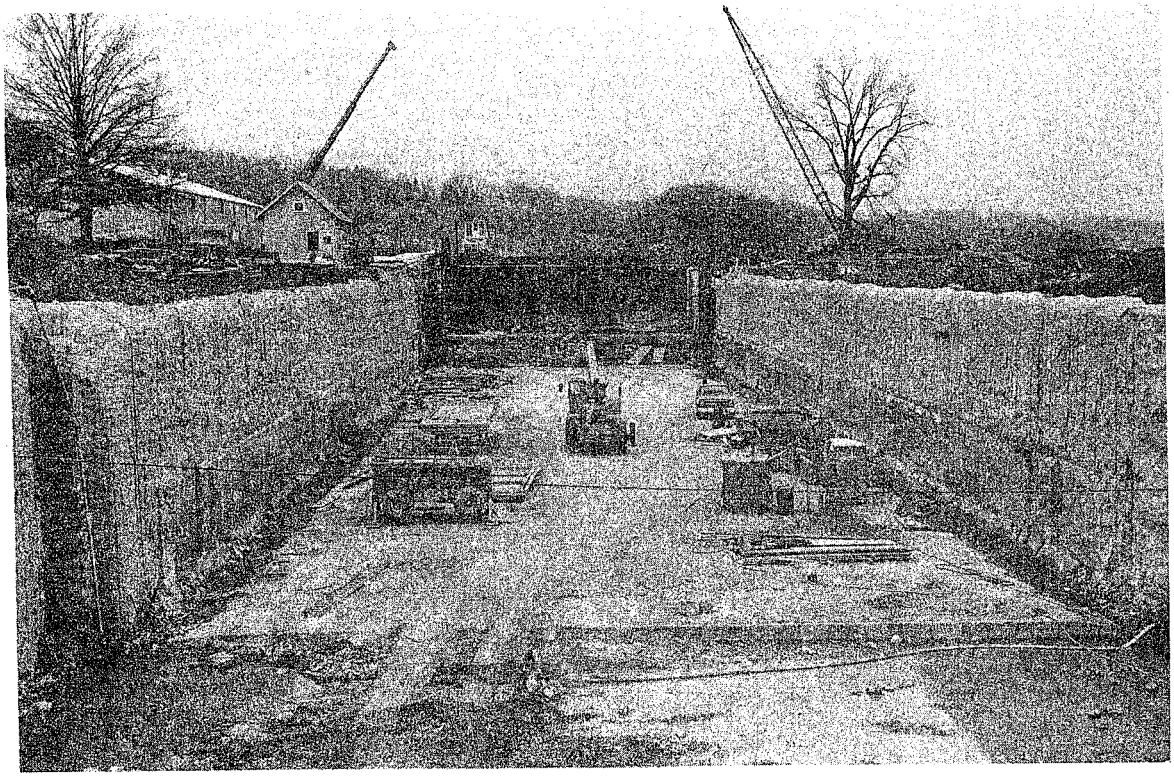


Figure 22. Lock walls following removal of deteriorated concrete, April 1980, Old Lock No. 14, Mississippi River

54. The replacement concrete was anchored to the existing concrete by No. 6 dowels spaced 4 ft on centers. The dowels were grouted into the existing concrete using prepackaged polyester resin grout cartridges. A reinforcing mat, No. 6 bars on 12-in. centers each way, was placed near the face of the replacement concrete for crack control. Forms for the new concrete were attached to the existing wall (Figure 23).

55. A concrete mixture proportioned with crushed dolomite coarse aggregate (1-in. maximum size), natural quartz fine aggregate and a water-cementitious ratio of 0.44 for a 3-in. slump, 5 percent air content, and a compressive strength of 4,000 psi at 28 days was used in the lock rehabilitation. Concrete mixture proportions, based on a 1-cu yd batch, were as follows:

Material	Weight, lb
Portland cement, type I	510
Fly ash, type F	110
Fine aggregate	1,347
Coarse aggregate	1,560
Water	375
Air-entraining admixture	--
Water-reducing admixture	--

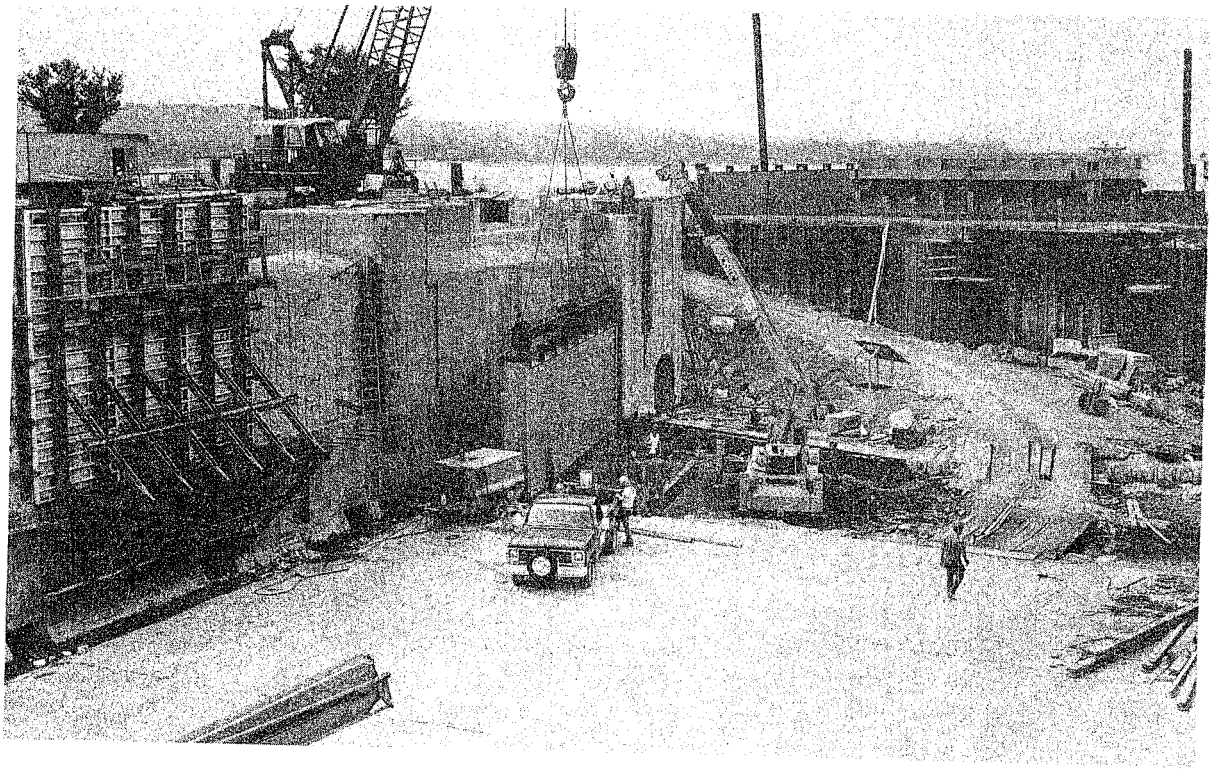


Figure 23. Concrete forming system and gate installation, August 1980, Old Lock No. 14, Mississippi River

56. The concrete was batched, usually 7 cu yd at a time, from a semi-automatic batch plant and transported to the jobsite, a distance of about 6 miles through urban areas, in transit mixers. Concrete placement was by crane and bucket using varying lengths of elephant trunk. The concrete was cured using pigmented curing compound. Very few shrinkage cracks were noted in the replacement concrete (Rock Island District 1980). The bid price for the cast-in-place concrete was $325 per cubic yard.

57. A testing program was conducted by the Rock Island District to evaluate the use of accelerated strength tests to predict the potential strength of concrete containing fly ash. For the accelerated tests, 6- by 12-in. test cylinders were prepared and cured in accordance with ASTM-C-684-74, Procedure A. A reusable steel mold having machined plates which were securely connected at both top and bottom of the mold was used for casting the 24-hr test cylinder. Immediately after molding, the test cylinders were placed into a curing tank. The temperature of the water at the time of immersion and throughout the curing period was 95° ± 5°F. After curing for 23-1/2 hr ± 30 min, the cylinders were removed from the curing tank and demolded. They were then capped and tested at 24 hr ± 15 min.

Paraffin-coated, single-use molds conforming to ASTM-C-470-76 were used in the field for casting the 7- and 28-day test cylinders. The 6- by 12-in. test cylinders were prepared and cured in accordance with ASTM-C-31-69.

58. Results (Burke 1981) confirmed that accelerated strength testing was reliable for quality assurance monitoring of concrete containing fly ash and for predicting the 28-day strength of concrete under field laboratory conditions. Based on these tests, the Rock Island District has adopted accelerated concrete strength testing for quality assurance at subsequent rehabilitation projects.

59. The lock was reported to be in excellent condition during the fourth periodic inspection, August 1984, although a number of cracks were evident in the replacement concrete (Figure 24).

Dresden Island Lock

60. The Dresden Island Lock and Dam is located immediately downstream of the confluence of the Des Plaines and Kankakee Rivers and is located at mile 271.5 of the Illinois Waterway near Morris, Illinois. The lock has a usable chamber of 110 ft by 600 ft with miter gates at both ends. Normal lift is 21.75 ft. The lock walls are concrete gravity type founded on rock (Figure 25). The upper guide wall consists of concrete piers with a concrete beam at the top of the piers. The upper guide wall is free standing with water at upper pool on both sides of the wall. The lower guide wall is a concrete gravity wall and retains backfill from the landside. The upper miter gate sills are concrete arches, and the lower sill is a thin concrete paving over the foundation rock. The dam includes an overflow spillway, tainter gates, ice chute, head gates, and a concrete arch. The arch is constructed over what was to be the sill of the smaller navigation lock which was never built. The project was completed in 1933.

61. In 1954, deteriorated portion of the lock chamber walls and the gate bays of the upper and lower service gates were resurfaced. Resurfacing in the lock chamber (Figure 26) extended over the even numbered monoliths 12 through 20 for the land wall and over the odd numbered monoliths 11 through 29 for the river wall. Also, portions of the back side of the river wall were resurfaced. Resurfacing consisted of removal of weathered concrete and refacing with anchored and reinforced shotcrete (Gunite).

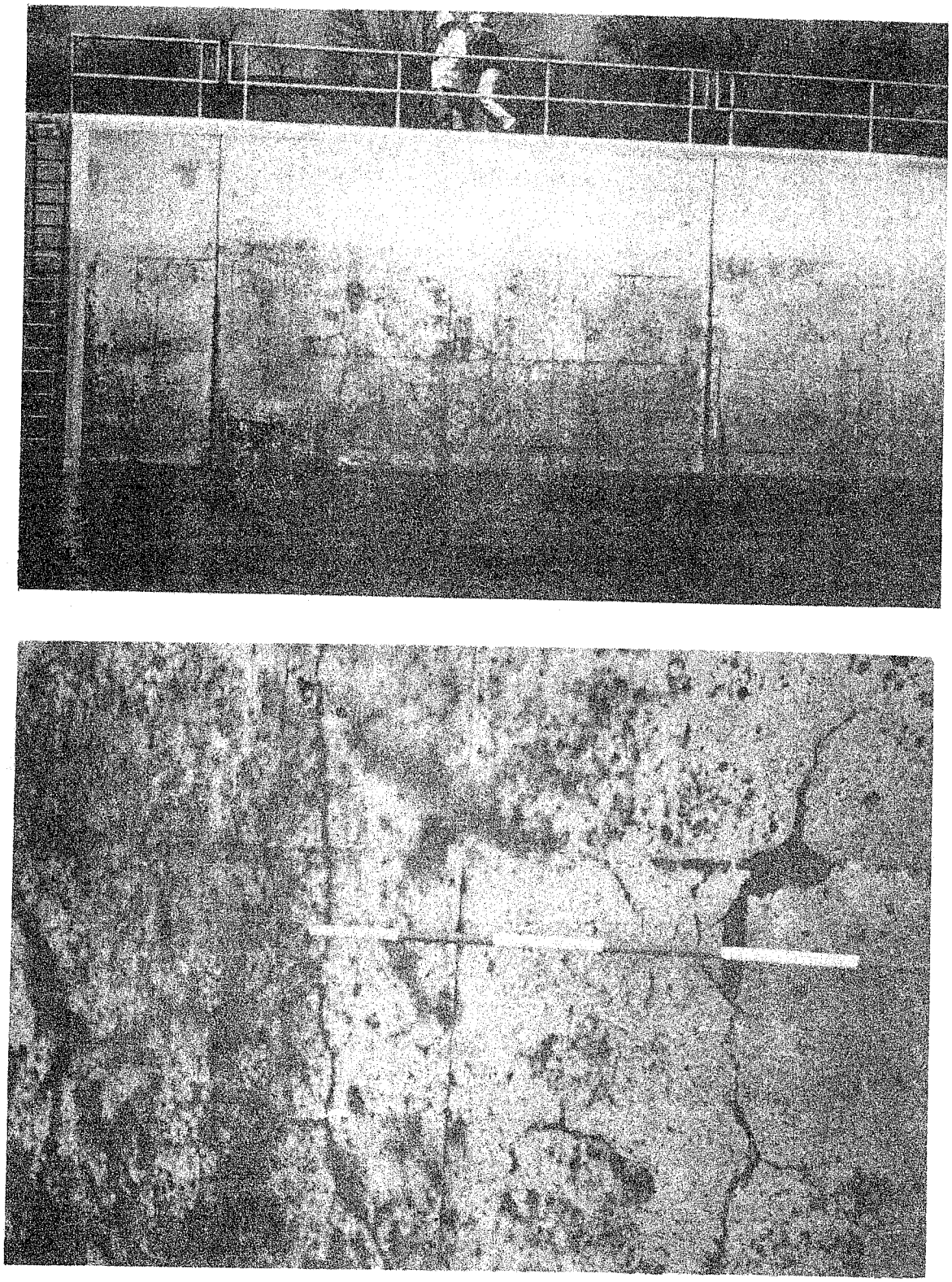


Figure 24. Typical concrete condition, August 1984, Old Lock No. 14, Mississippi River

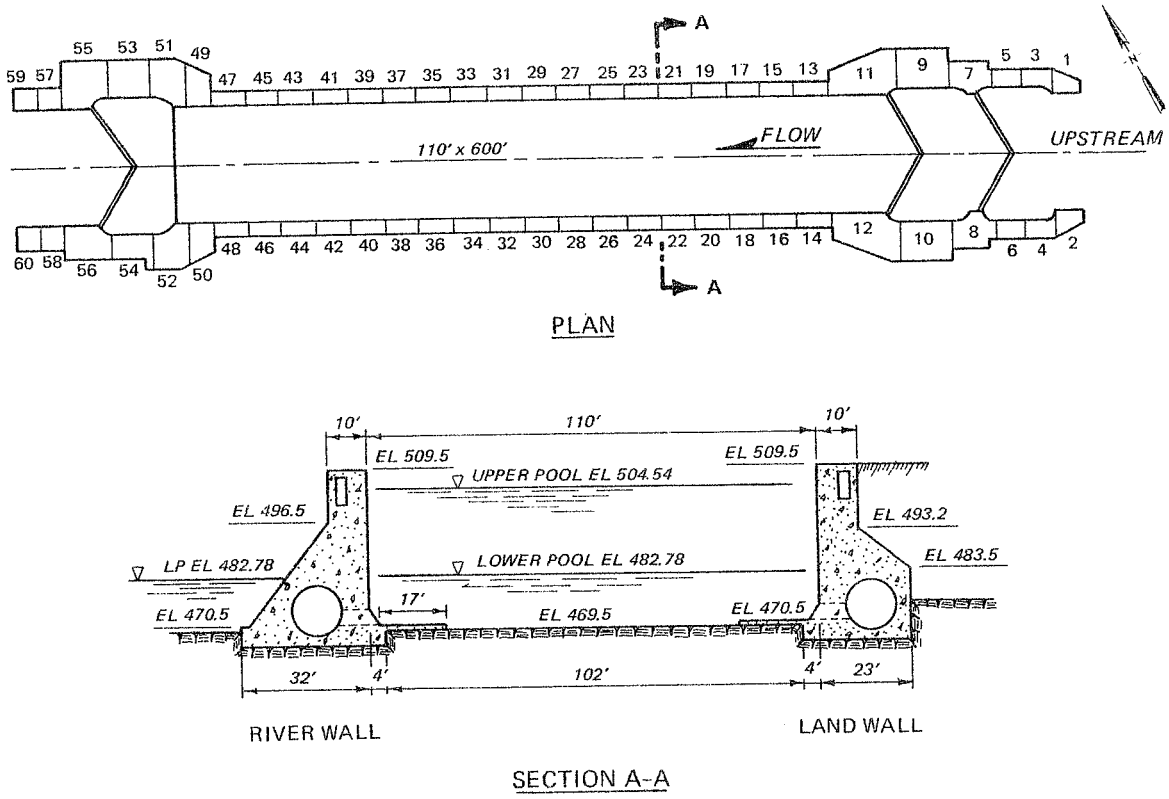


Figure 25. Plan and typical section, Dresden Island Lock

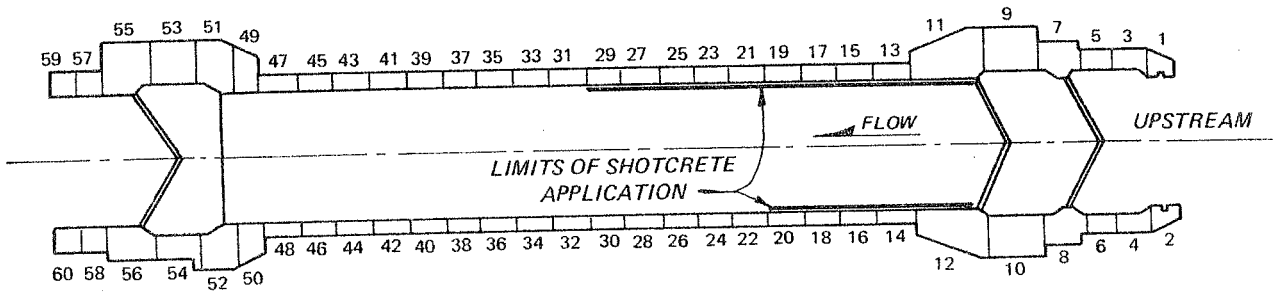


Figure 26. Wall monoliths that were refaced with shotcrete, 1954, Dresden Island Lock

62. A minimum of 4 in. of concrete was removed back of the finished faces of walls at the work locations. Where existing concrete walls had wholly or partly disintegrated surfaces, the finished faces were reestablished by means of bronze wire lines stretched between the sound areas of the walls. Following concrete removal, surfaces were sounded and loose or shattered areas of concrete were removed to solid concrete.

63. One layer of anchored wire mesh reinforcement (4 by 4 in., No. 6) was used for each layer of shotcrete applied. The anchor bolts for the wire mesh were 5/8-in. diameter with 2-in. right angle bend at the outer end and were spaced 24 in. on centers vertically and horizontally. In addition, when the depth of concrete removal from the finished face of the wall was 18 in. or greater, bar reinforcement doweled into the existing concrete was installed. Where successive layers of mesh were required, each layer was fastened to the same set of anchor bolts with No. 16 or No. 18 iron wire.

64. The concrete surface to receive the shotcrete was thoroughly cleaned and washed with water and compressed air to remove all dust, dirt, and other foreign materials. The prepared surface was moistened not more than one hour prior to placement of the shotcrete, then scoured with an air and water jet and finally with an air jet alone to remove all traces of water.

65. The shotcrete, approximately 1 part cement to 3-1/2 parts sand by dry volume, was mixed in a stationary mixer and transported by conveyor to the shotcrete apparatus (Figure 27). The shotcrete nozzle was held at a distance of 3 or 4 ft from the surface and at such a position as to direct the flowing material at approximately right angles to the surface being covered. At the end of a day's work, the shotcrete was sloped off to a thin, clean, regular edge, at approximately a 45-deg slope. No square joints were permitted. Shotcreting was carried upward from the bottom of each monolith to the top. Shotcreting at a particular level was carried back and forth across the shooting platform as deemed necessary to avoid slump. Where the average thickness for the area being worked was 5 in. or less, the wall was built out to the finished face without waiting for the shotcrete to set. In large pockets, a layer of mesh was used for each layer of shotcrete, and each layer was allowed to set at least 3 hr before successive layers were applied. The thickness of a layer did not exceed 5 in. except in certain limited areas where in a series of successive layers, one layer not greater than 6 in. in thickness was allowed. Shotcreting through more than two layers of mesh was not allowed.

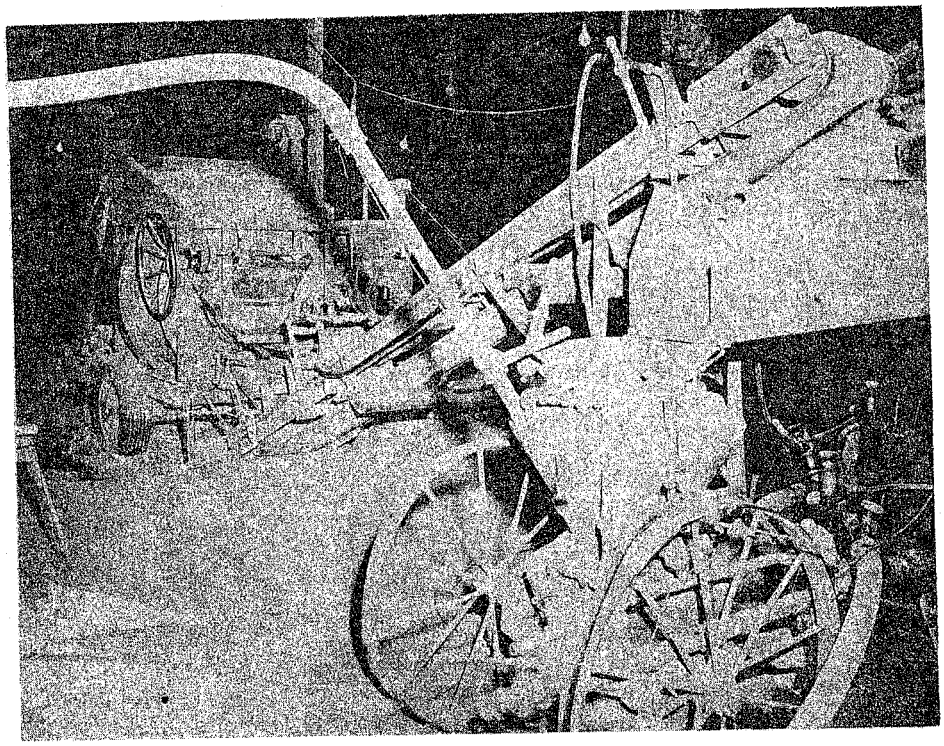


Figure 27. Shotcrete mixing and delivery equipment, 1954, Dresden Island Lock

66. After shooting to within 1/4 in. of the repaired face of wall, high spots in the shotcrete were cut off with the edge of a steel trowel. Upon completion of an entire monolith, the surface was sealed with a flash coat not more than 1/4 in. thick, except where additional shotcrete was required to bring any remaining low areas to grade. No projection of the refaced wall beyond the established finished face was allowed. The finished face of any wall was not more than 1/4 in. back of the established face. Curing was accomplished by means of an approved pigmented curing compound of the surface membrane type. The curing compound was applied by power spraying equipment as soon as free water disappeared.

67. All work was performed in a 30-day period during the winter when the lock was shut down. The maximum number of men practical was used working three 8-hr shifts. During the peak construction period, 190 men were engaged in the work. Cost of the resurfacing was $158,000. The lock chamber immediately prior to flooding following the repair is shown in Figure 28. The shotcrete resurfaced monoliths can be seen in the background.

68. During the original lock construction, horizontal surfaces were finished without any slope. As a result, water ponding on these surfaces

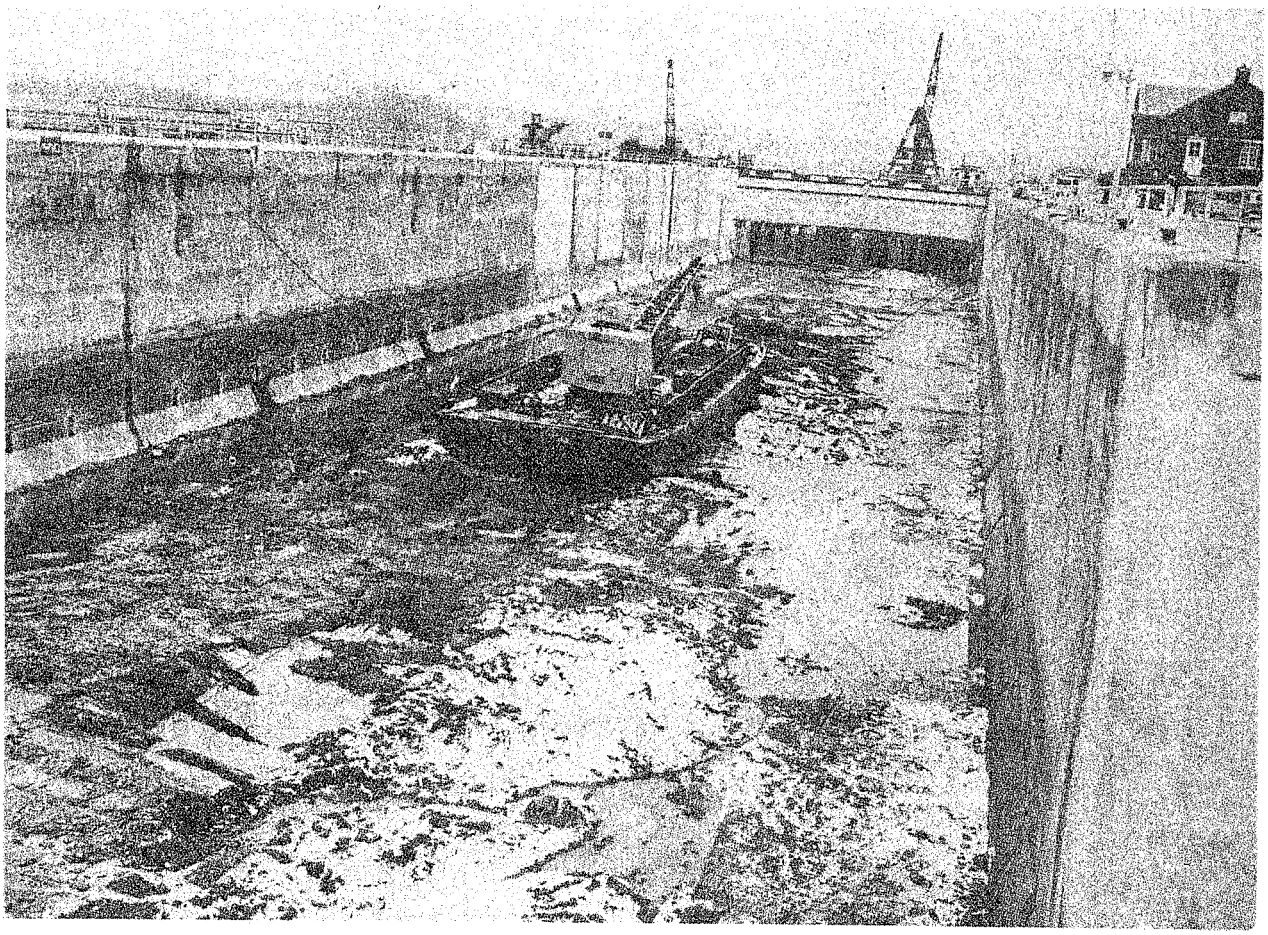
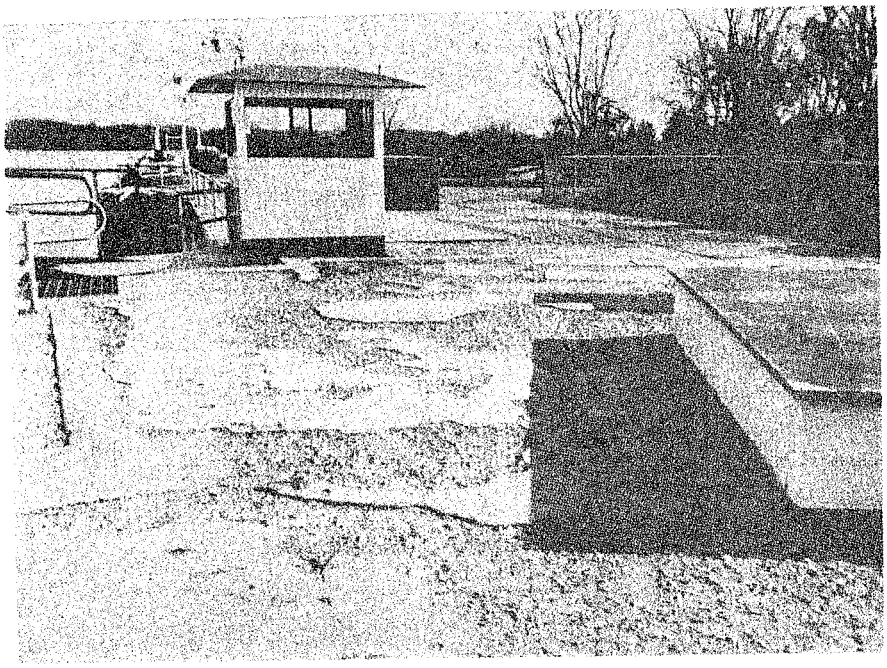


Figure 28. Chamber immediately prior to reflooding after 1954 repairs, Dresden Island Lock

contributed to the concrete deterioration through cycles of freezing and thawing (Figure 29). In 1961, the top surfaces and the upper 3 ft of the vertical walls of the lock were resurfaced. This repair involved removing a minimum of 4 in. of concrete and replacing with new air-entrained concrete reinforced with wire mesh. In addition, corner armor was added to the lock chamber walls.

69. During the second periodic inspection in August 1972, random cracking of the resurfaced concrete on top of the lock walls was observed. This cracking was attributed in part to reflective cracking from the original concrete. Concrete deterioration to depths of about 8 in. was reported for those portions of the lock chamber walls which had not been resurfaced. The concrete and shotcrete lock chamber resurfacing was reported to be in excellent condition except for spalling at the monolith joints. Introduction of expansion joint material at lock wall monolith joints during resurfacing was



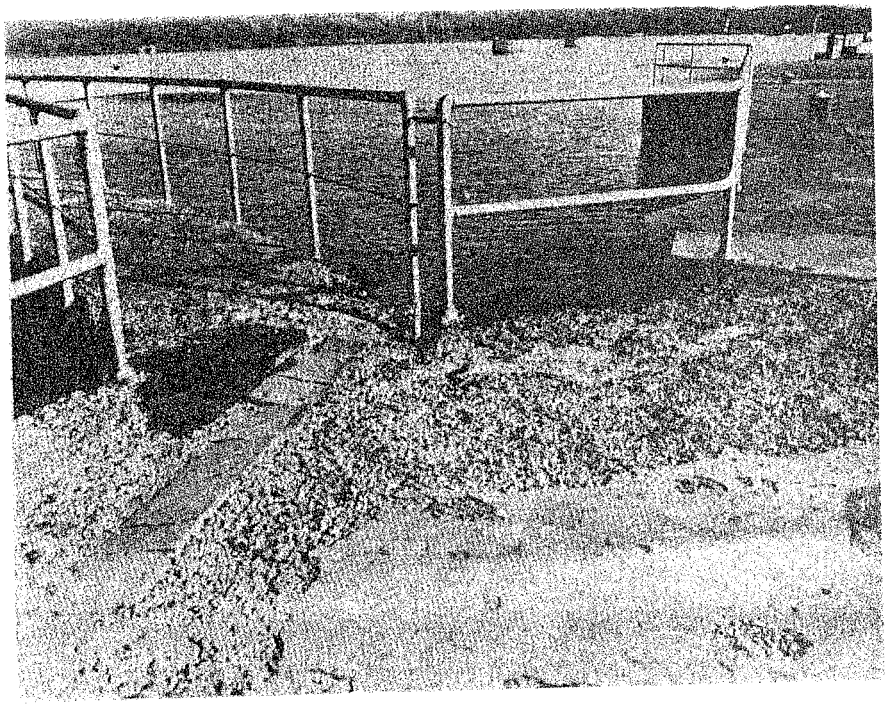


Figure 29. Typical concrete deterioration on tops of lock walls, 1961, Dresden Island Lock and Dam

believed to have contributed to spalling at the joints. Since there are no expansion joints in the lock walls, the expansion joint in the resurfaced zone cannot function and only absorbs water which causes spalling through cycles of freezing and thawing. It was recommended that expansion joints not be used in future resurfacing projects.

70. An extensive core drilling and laboratory testing program was conducted during 1976 and 1977 to ascertain the extent and cause of concrete deterioration (Stowe et al. 1980a). Four drill holes were located in areas that had been resurfaced with concrete in 1961. The new concrete was found to be structurally sound by itself, but in certain locations it was considered susceptible to barge impact because of the frost-damaged concrete beneath (Figure 30). The original concrete in the lock walls was nonair-entrained

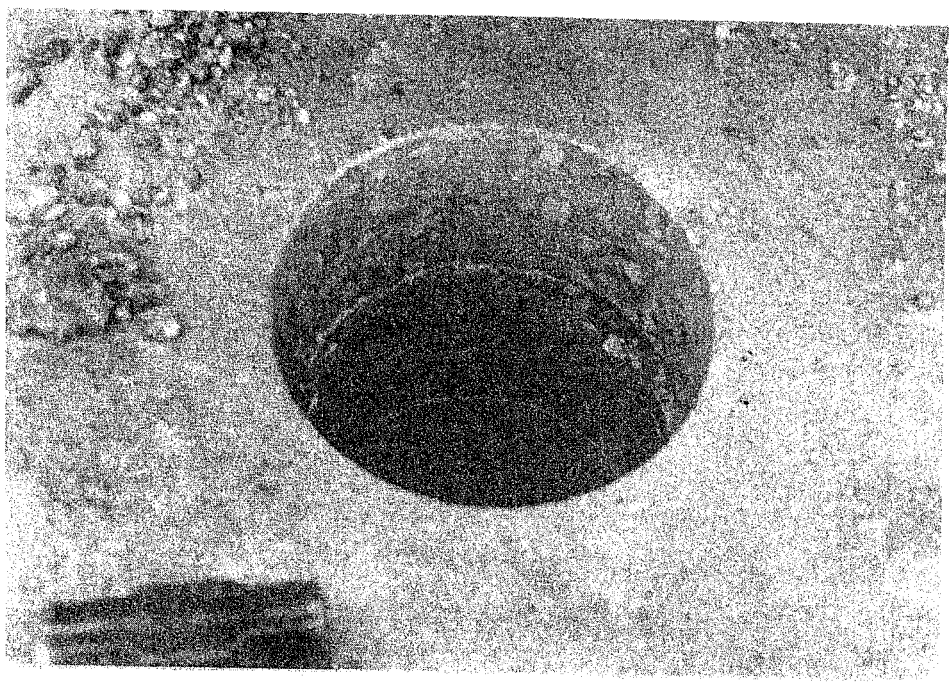


Figure 30. Downhole view showing newer good quality concrete overlay and underlying deteriorated concrete, 1977, Dresden Island Lock

concrete that was well consolidated during placement. It was structurally sound in areas which had not been affected by frost action.

71. Average physical properties, as determined from tests on concrete core taken from the land wall, were as follows:

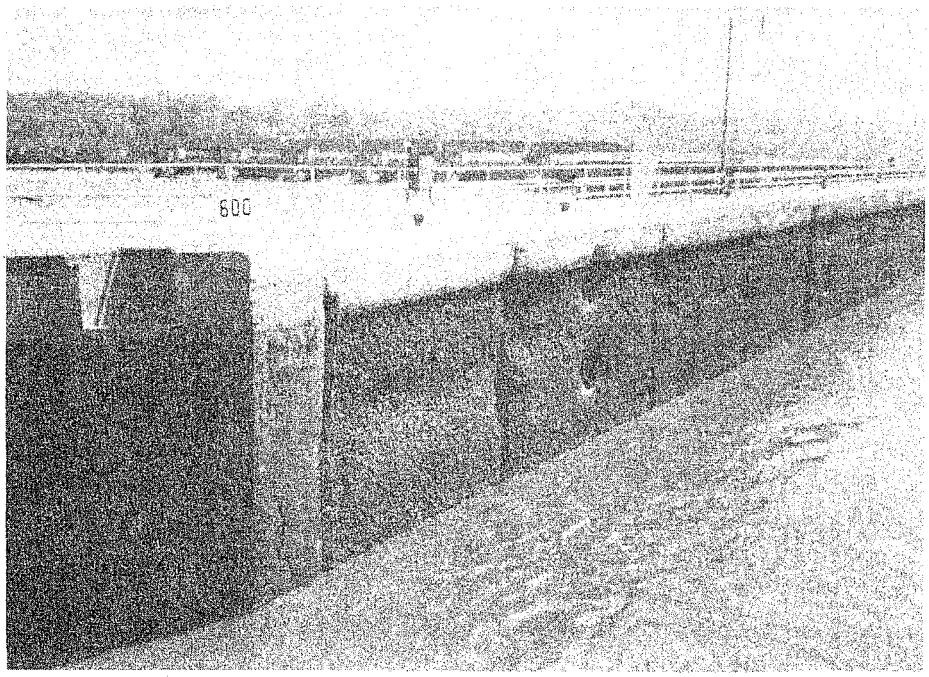
Test	Results	
	Near Surface Concrete	Bottom of Core Specimens
Effective unit weight, pcf	149.7	150.0
Compressive wave velocity, fps	13,438	15,258
Compressive strength, psi	5,840	5,860
Modulus of elasticity $\times 10^6$, psi	2.75	4.57
Poisson's ratio	0.13	0.21

The compressive strength and unit weight of the surface concrete were essentially the same as that of concrete at greater depths. However, results of the remaining tests were lower for near surface concrete as compared to concrete at greater depths. This difference is attributed to microscopic cracking in the near surface concrete.

72. Deterioration of the exposed and near surface original concrete ranged from light to severe. About 80 percent of the exposed vertical surfaces of the concrete in the lock walls had been effected by frost action (Figure 31). The average depth of concrete deterioration as determined by petrographic examination were as follows: lock chamber walls, 0.7 ft; riverside of river wall, 0.9 ft; and the upper gate bays, 1.5 ft. Severe loss of concrete was evident at most monolith joints.

73. Three borings were drilled from inside the lock chamber into the river wall section previously refaced with shotcrete. The shotcrete had a minimum thickness of 12 in. (Figure 32) and exhibited excellent bond to the original concrete. Air-void data determined according to CRD-C 42 indicated the shotcrete had about 3 percent total air with approximately 2 percent of it in voids small enough to be classified as useful for frost resistance. The air-void spacing factors ranged from 0.010 to 0.014 in. While these values are larger than is desirable (0.008 in. is considered the maximum value for air-entrained concrete), they may have imparted some frost resistance. Also, there were no large voids or strings of voids due to lack of consolidation such as have been observed with other shotcrete specimens. Typical surface conditions of the shotcrete are shown in Figure 33.

74. A major rehabilitation program for Dresden Island Lock and Dam was developed by the Chicago District in 1977. The condition of existing structures, need for rehabilitation, and proposed rehabilitation design were described in a Design Memorandum (Chicago District 1977). Major features of



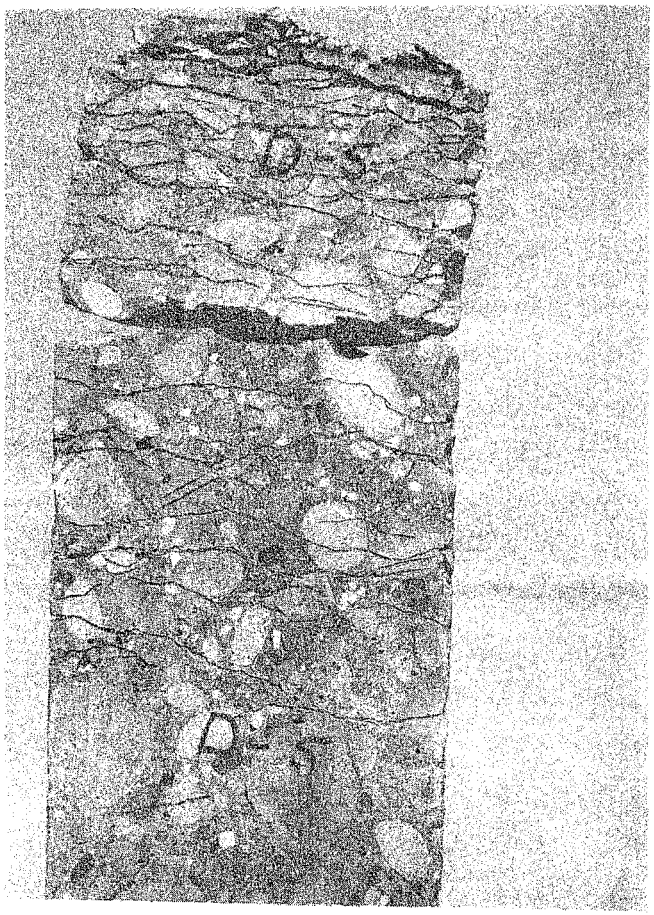


Figure 31. Typical concrete deterioration, 1977, Dresden Island Lock

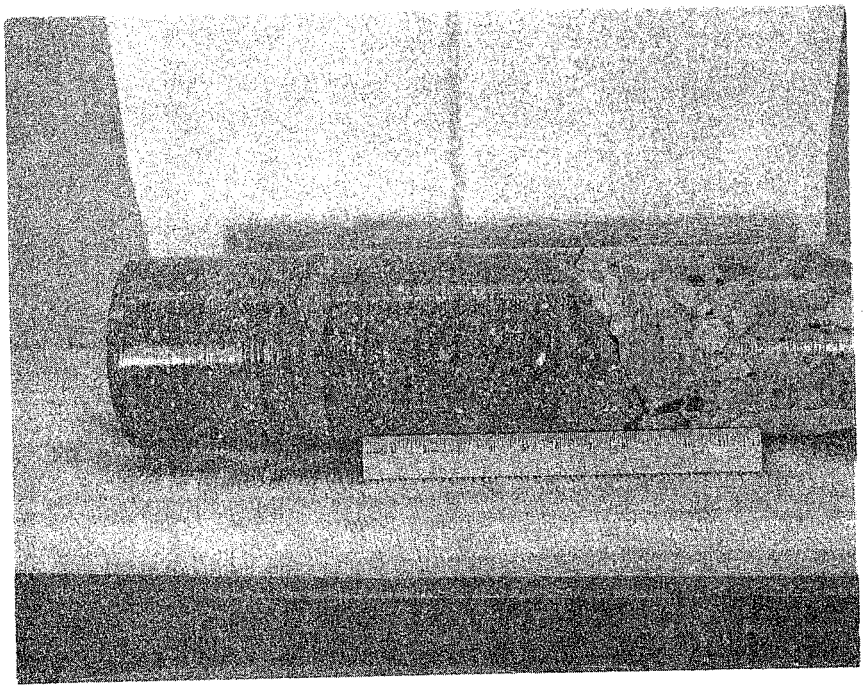
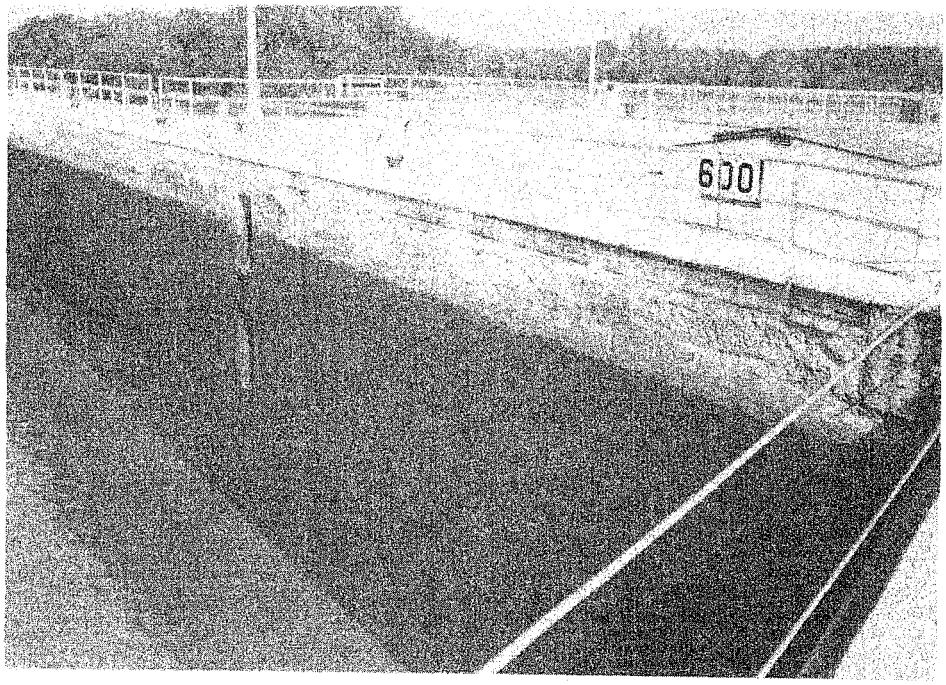


Figure 32. Horizontal core taken through shotcrete resurfacing, 1976, Dresden Island Lock

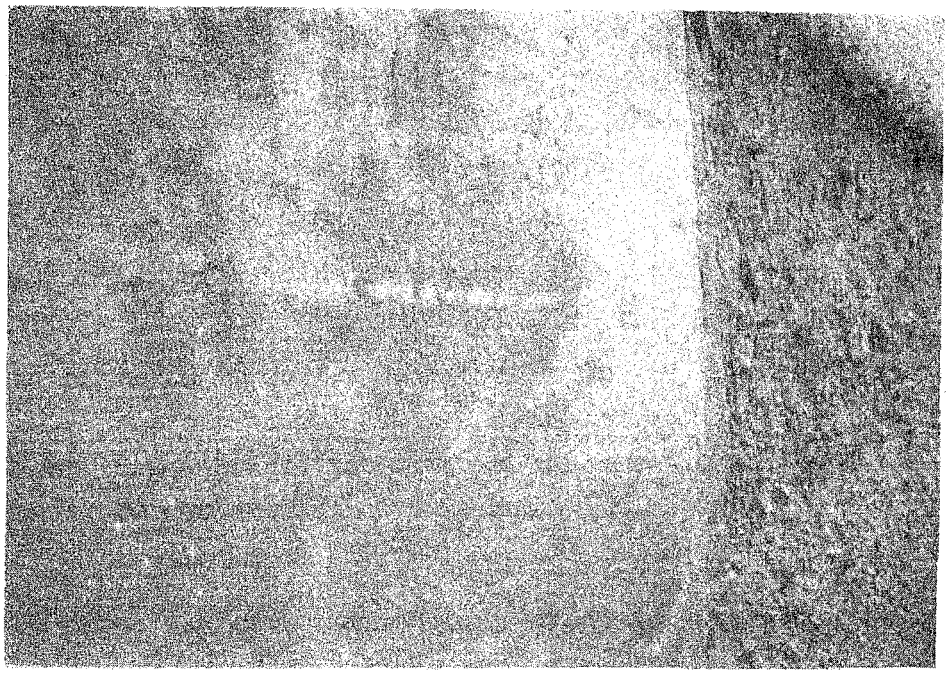
the rehabilitation included: (a) resurfacing of lock chamber walls, upper and lower gate bays and forebays, back side and top of river wall, and stairs at lower end of lock; (b) repairs to upper and lower service gates, invert of lock culverts and miter gate machinery, and stabilization of lower guide wall; (c) replacement of electrical system; (d) resurfacing of the tainter gate piers of the dam and head gate section; (e) reconstruction of counterweight of one tainter gate; (f) repair of ice chute abutments, walkway bridge, and tainter gate seals; (g) installation of heating-deicing units for two gates; and (h) construction of scour protection below the tainter gates. Total cost of the rehabilitation was estimated at $8,300,000.

75. A contract to accomplish items (a) and (b) above was advertised in January 1978, and a contract was awarded in April 1978 to the low bidder, J. M. Foster, Incorporated, of Gary, Indiana, in the amount of $4,444,444. It was anticipated that the majority of the work covered under this contract would be completed during a two-month shutdown of the lock scheduled to begin 1 August 1978.

76. The contract required removal of deteriorated concrete to a depth of approximately 15 in. from the face of the lock wall prior to resurfacing; however the contractor elected to remove concrete to a depth of 17 in. to avoid complications in removal at the top of the wall. Height of concrete



a. River wall



b. Interface between shotcrete (left) and original concrete

Figure 33. Shotcrete surface conditions, 1977, Dresden Island Lock

removal was about 28 ft as measured from the top of the lock wall. Use of explosives in the concrete removal was permitted. The contractor was required to submit a detailed blasting plan that included the diameter, depth, and spacing of the blast holes; the size, location, and type of charges; the blasting sequence; the monitoring plan; and the safety precautions to be followed. The contract limited the extent of blasting to one 30-ft-wide monolith at a time. To minimize the tensile stresses on the removal plane, the spacing of the blast holes was limited to 8-in. centers. Each blasting was to be monitored by three seismographs as a minimum. One seismograph was to be located at the lock control house, and two were to be at the gate block monoliths nearest to the blast located on opposite sides of the lock.

77. The contractor employed a subcontractor for blasting operations who in turn used the services of another firm for development of the blasting plan and monitoring of the blasting activities. Three test blasts were performed prior to the adoption of the plan for production blasting. The blasting plan for the three tests is shown in Figure 34. The test data are summarized as follows:

Test Blast Data

Test	No. of Holes	Total Explosive Weight, lb	Delay msec	Max. Explosive Weight/Delay lb
A	17	12.2	0, 2 to 6	3.56
B	18	15.75	0, 2 to 7	5.8
C	39	38.8	0, 2 to 9	8.3

Seismograph Data

Test	Seismograph No.	Particle Velocity in./sec	Energy Ratio	Scaled Distance ft/16-1/2
A	1	0.16	0.007	105.9
	2	0.30	0.025	92.7
	3	0.97	0.26	22.8
B	1	0.18	0.0089	87.2
	2	0.41	0.046	76.8
	3	1.04	0.30	18.7

(Continued)

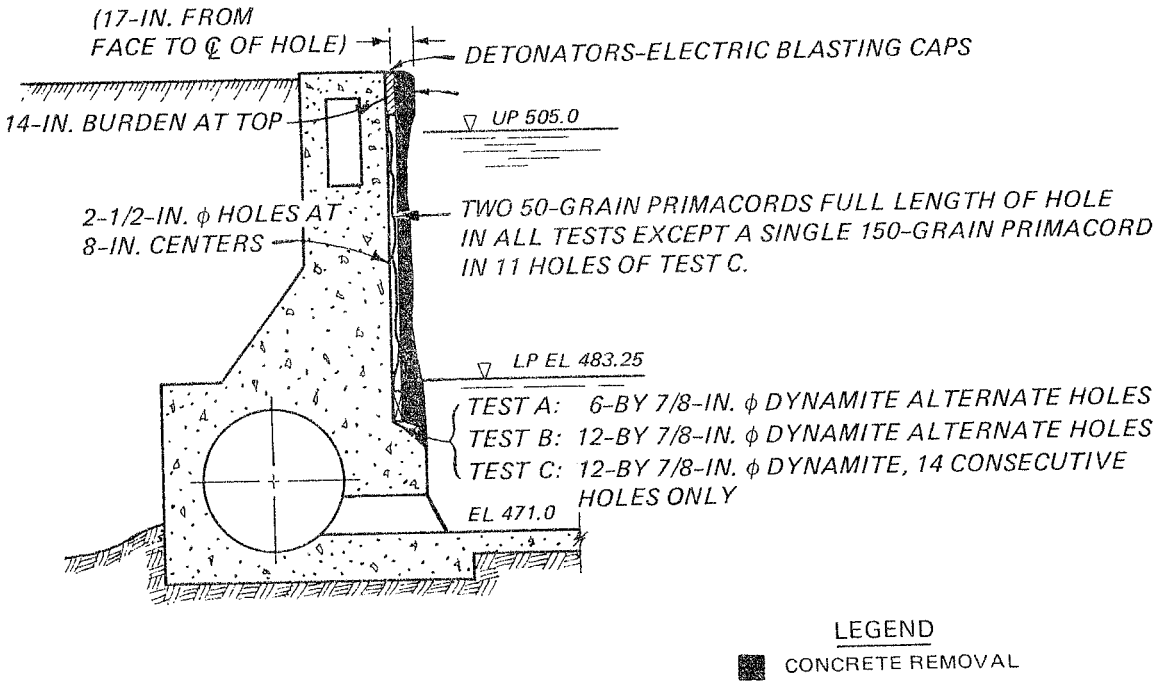
Seismograph Data (Concluded)

Test	Seismograph No.	Particle Velocity in./sec	Energy Ratio	Scaled Distance ft/16-1/2
C	1	0.07	0.0013	78.1
	2	0.39	0.042	67.7
	3	1.55	0.66	12.15

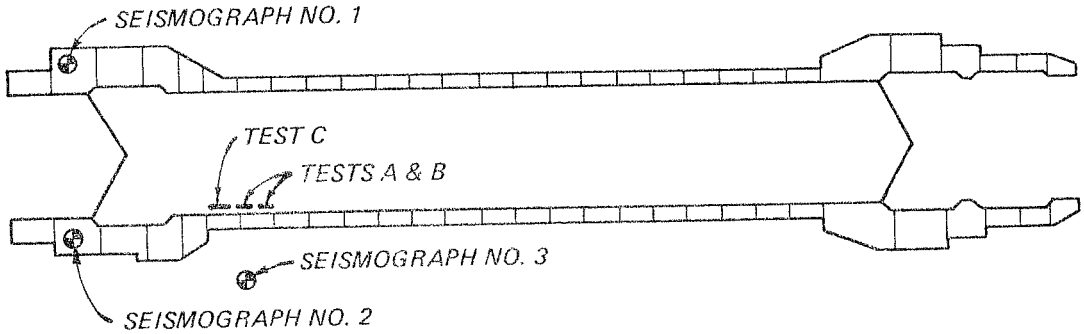
Typical conditions of the concrete in the lock wall following blasting are shown in Figure 35.

78. From the nine test points, a linear regression of particle velocity versus scaled distance was developed for guidance in production blasting (Figure 36). While all three tests produced satisfactory results, generally a clean break without cracking in the remaining structure, the size of charges and delays of test blast A were adopted for production blasting. Forty-five holes at 8-in. spacing, constituting the width of one monolith, were detonated at a time. The size of charges and delays are extremely important factors in concrete removal. In an almost identical lock resurfacing project, at Starved Rock, the contractor test blasted a 30-ft-wide monolith with 45 holes at 8-in. centers, each hole loaded with 150-grain detonating cord (no dynamite) and no delays. The results were disastrous with numerous cracks developing at corners of cable galleries and elsewhere. The contractor modified the blasting procedures by decreasing the detonating cord to 100 grain and using delays. Excellent results were obtained with the modified system (Juzenas 1980).

79. Following removal of concrete to a depth of 17 in. from the original wall surface, the lock walls were resurfaced as shown in Figures 37 and 38. Concrete anchors were No. 6 reinforcing bars spaced on 2-ft centers each way. The anchors were embedded using two-component polyester resin grout cartridges. Insertion of the anchor into the drill hole ruptured the seal between the two-component resin in the previously installed cartridge. Mixing of the grout was accomplished by spinning the anchor using an electric drill. A straight anchor bar was selected for ease in attachment to the drill chuck. The size, length of embedment, and spacing of the anchors were arbitrarily selected using engineering judgment. To assure adequate embedment of the anchors, pull-out tests were specified. Three anchors were specified to be



a. Type and location of explosives



b. Location of test blasts and seismographs

Figure 34. Test blasting plan, Dresden Island Lock

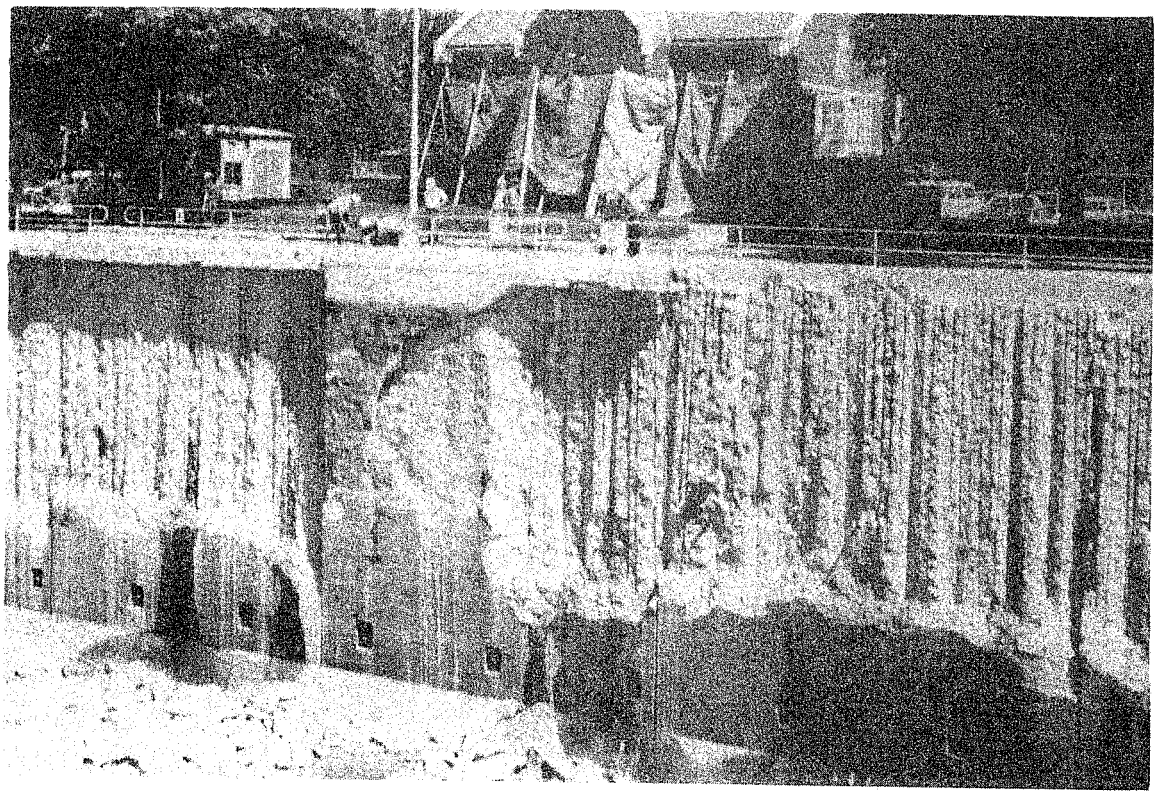


Figure 35. Typical concrete surface conditions after blasting, Dresden Island Lock

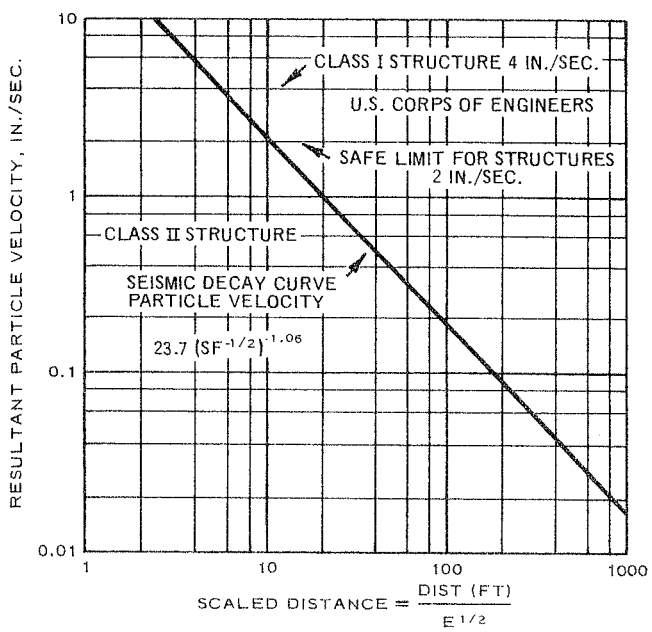


Figure 36. Resultant particle velocity versus scaled distance, Dresden Island Lock

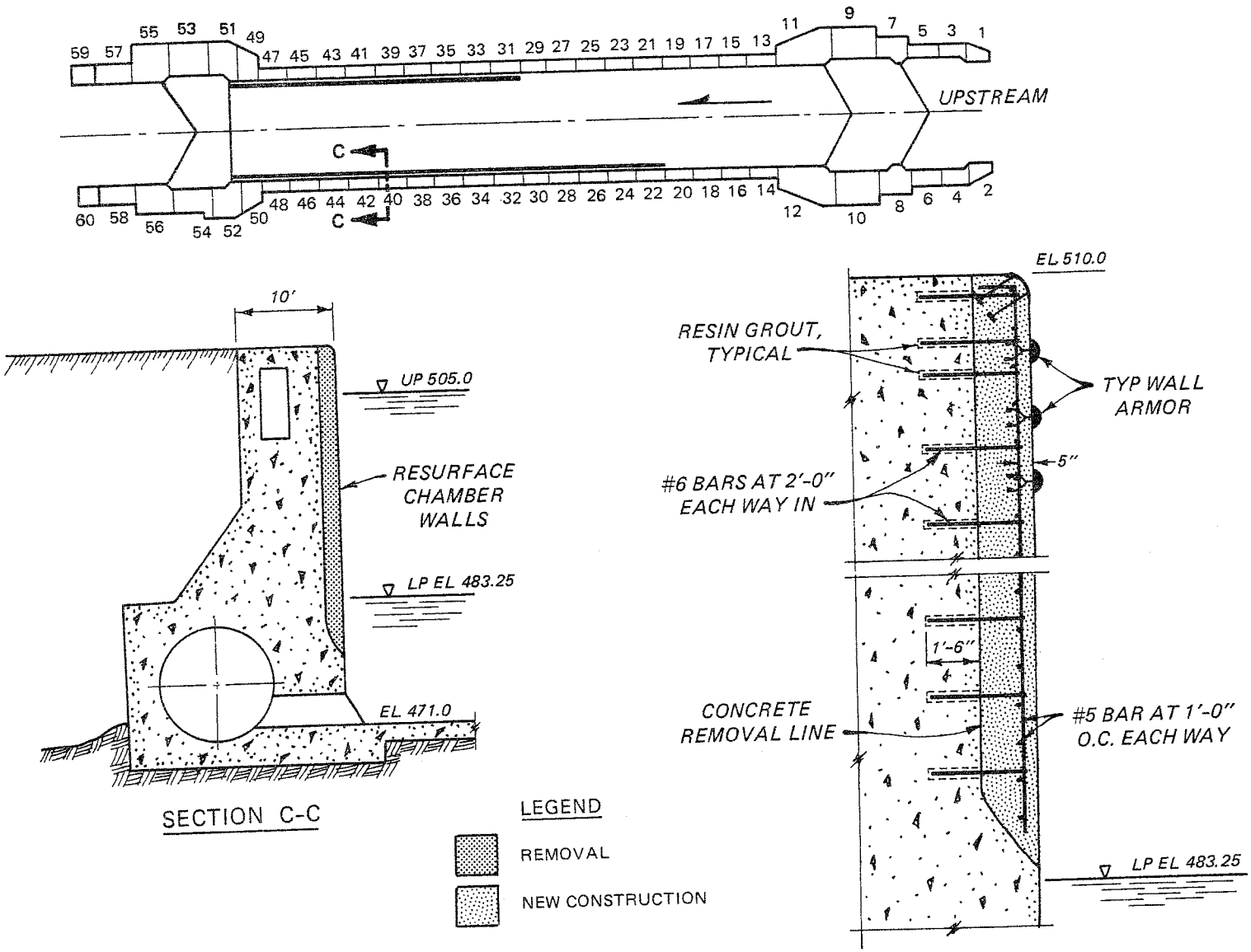


Figure 37. Land wall resurfacing, Dresden Island Lock

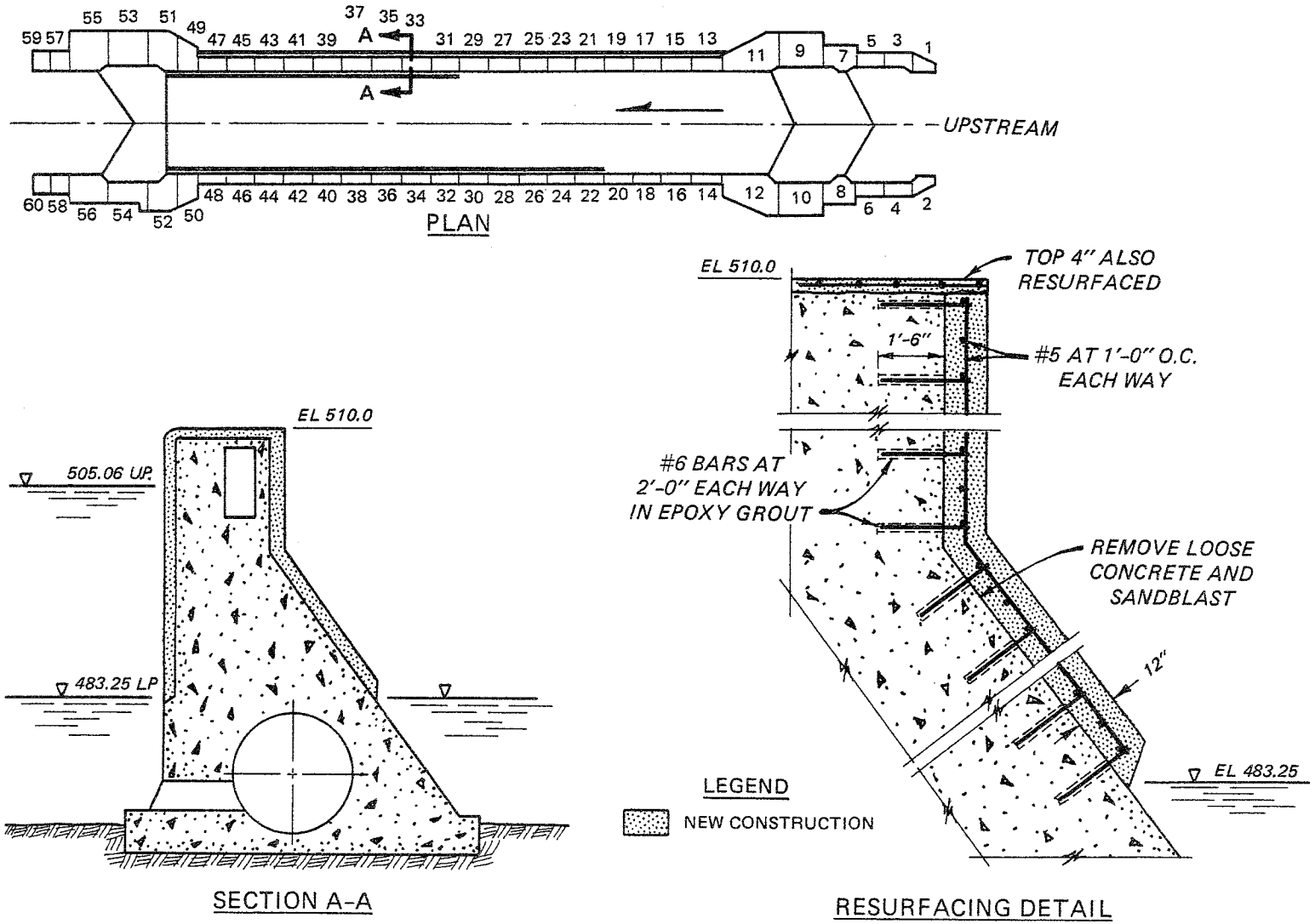


Figure 38. River wall resurfacing, Dresden Island Lock

tested initially and 2 percent of all the anchors thereafter. Specified anchor pullout load was 8 tons, which is equal to 90 percent of the yield strength of the anchor of grade 40 reinforcing steel. None of the bars failed under the test load. Embedment of the anchor bars into the new concrete was about equal to that required for development of a tension bar.

80. The concrete reinforcement, No. 5 bars on 12-in. centers each way, was arbitrarily selected. The reinforcement was somewhat more than specified in EM 1110-2-2103, "Details of Reinforcement-Hydraulic Structures," which requires, for slabs restrained on one face, reinforcement to be equal to 0.20 percent of gross cross-sectional area half in each direction, placed near the unrestrained face. The Guide Specification (CW-03301) for Cast-in-Place Structural Concrete was used in preparation of contract specifications. Compressive strengths of 4,000 psi at 28 days were specified. To minimize shrinkage cracking, a water-cement ratio of 0.5 maximum was specified. The drawings specified a construction joint at the midheight of the 28-ft resurfacing or, as an alternative to a joint, required the contractor to submit a concrete vibrating plan. The contractor chose the latter.

81. In the earlier shotcrete resurfacing, 1/2-in.-thick expansion joint material was placed at the monolith joints. At the time of the concrete resurfacing, the monolith joints exhibited considerable spalling. It was believed that the expansion joint material becomes saturated and, through cycles of freezing and thawing, contributed to spalling at the monolith joints. Based on this observation and the fact that the existing monolith joints were tight, only asphalt saturated felt paper (30-lb weight) was used in the monolith joints to serve as a bond breaker.

82. Wall armor was installed in that portion of the lock walls previously repaired by shotcreting as shown in Figure 39. Note that at the bottom removal line drilling and broaching was specified to prevent cracking in the concrete below. Similar armor was installed in the concrete resurfacing.

83. The 12-in. thick concrete overlay on the backside of the river wall was placed over the existing weathered concrete (Figure 38). The new concrete was intended to serve as a sealer and to protect the concrete below from further disintegration. The existing weathered concrete was not removed for reasons of economy. It was believed that with anchored concrete, where structural and user considerations permit, this method may be the optimum repair

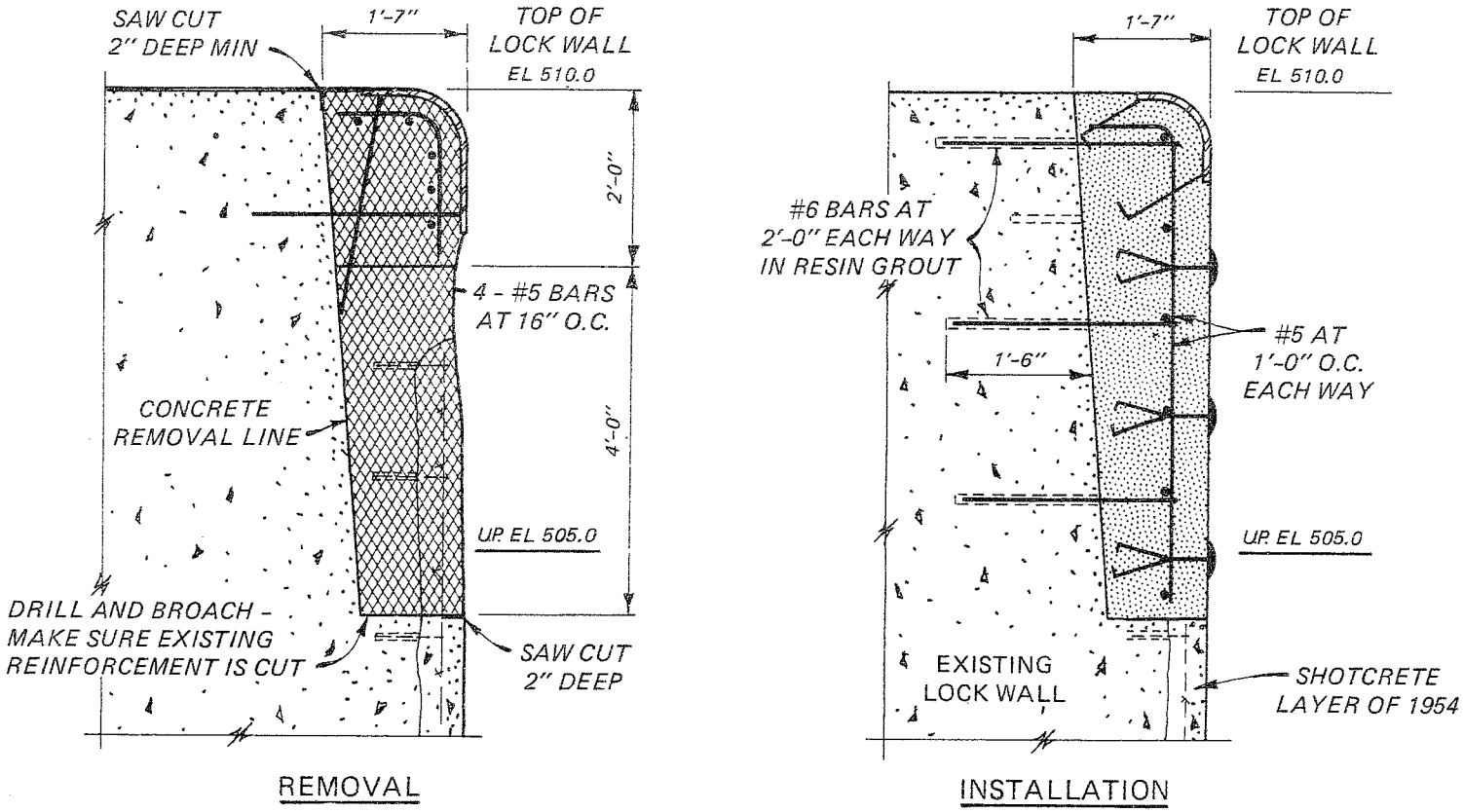


Figure 39. Armor installation in portions of lock wall previously resurfaced with shotcrete, Dresden Island Lock

solution. This repair section will serve as a prototype test on durability of this type of resurfacing.

84. Calculations using conventional methods showed that the lower guide wall was usable under normal operating conditions. The fact that the wall had stood for about 40 years while the lock was in operation appeared to say something about the reliability of the analysis when performed by conventional methods. However, when analyzed by the finite element method, the wall was found to be stable with about 25 percent of the base in compression. Stabilization of the lower guide wall by posttensioned tendons was considered necessary to increase its stability against overturning. The anchors were nine-strand, seven-wire anchors, designed for 225 kips working load (Figure 40). They were spaced at about 7-1/2-ft centers. The anchors were about 48-ft long with a bonding length in rock of about 25 ft. The anchors were embedded in grout their full length.

85. Shrinkage cracks in the replacement concrete were observed during the resurfacing operation. The cracks were about 4 to 8 ft apart and generally ran in both horizontal and vertical directions. Since such cracking may accelerate concrete disintegration, the cracks were considered undesirable. It was suggested that fiber-reinforced concrete or shrinkage compensating cement might be beneficial in eliminating such cracking (Juzenas 1979).

86. Although there is some minor spalling at monolith joints and at the interface between the shotcrete and the concrete placed in 1978 when horizontal armor was installed (Figure 41), overall the shotcrete remains in excellent condition after more than 30 years in service. With the exception of some isolated spalling (Figure 42), the concrete resurfacing inside the lock chamber is in good condition. Cracking previously reported in the replacement concrete has generally been obliterated by staining on the concrete surface. The concrete overlay on the back side of the river wall appears to be in generally good condition although there are a number of cracks in some areas (Figure 43). Minor seepage occurs sporadically through a horizontal cold joint located at midheight on the wall.

Lock and Dam No. 3, Monongahela River

87. Lock and Dam No. 3 is located on the Monongahela River in Allegheny County immediately upstream of the town of Elizabeth, Pennsylvania. It is

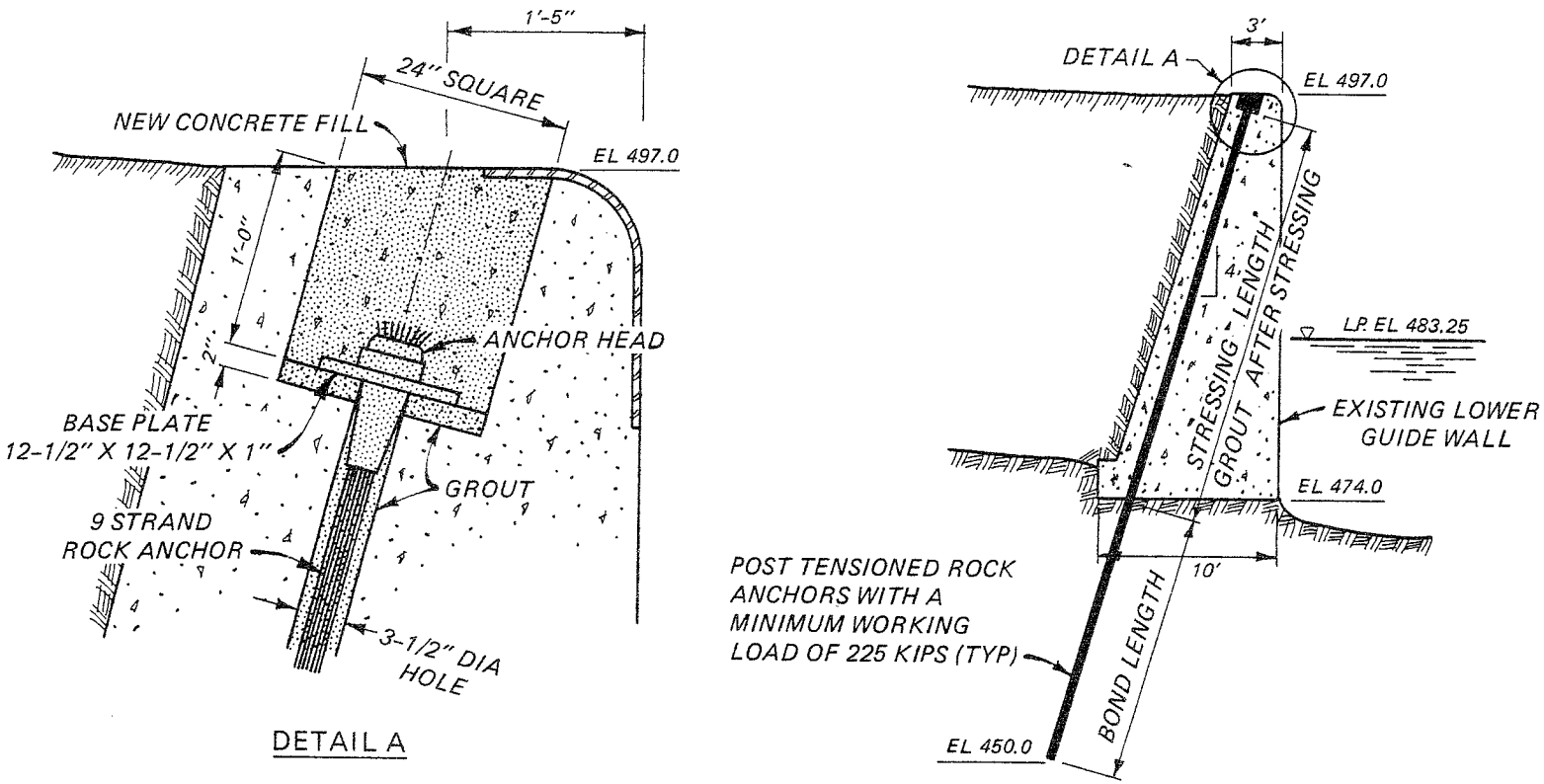
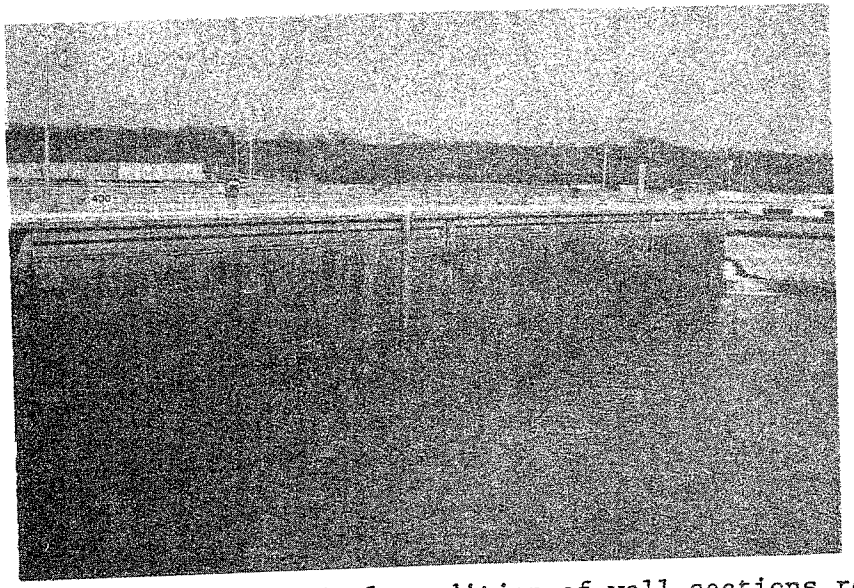


Figure 40. Lower guide wall stabilization, Dresden Island Lock



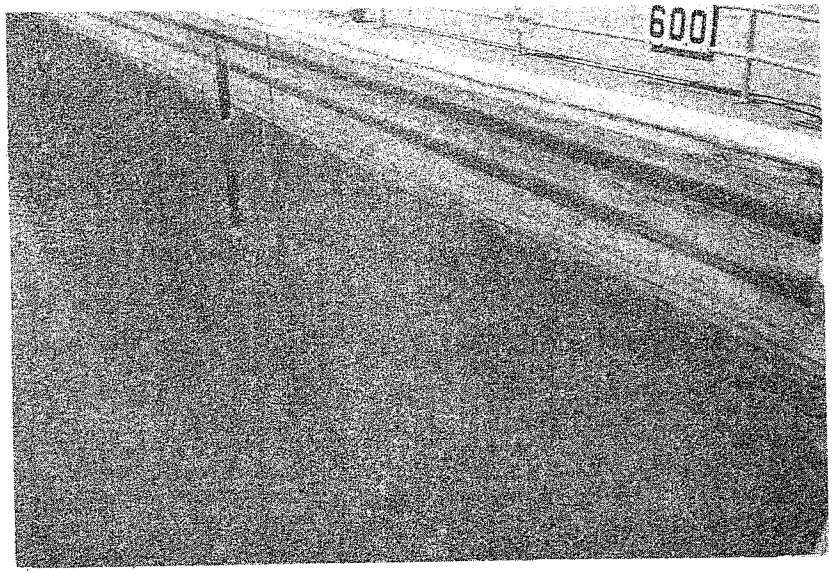


Figure 41. Typical condition of wall sections resurfaced with shotcrete, 1986, Dresden Island Lock

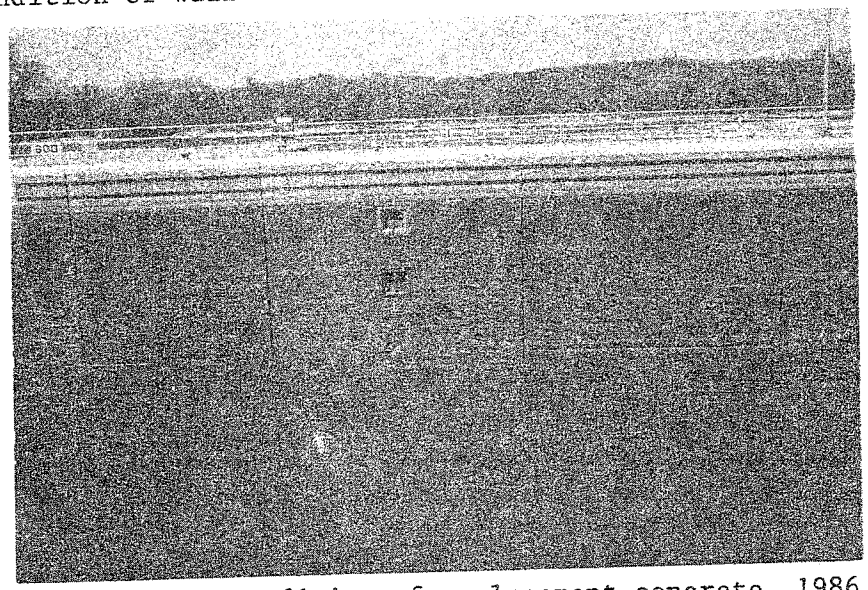


Figure 42. Condition of replacement concrete, 1986
Dresden Island Lock

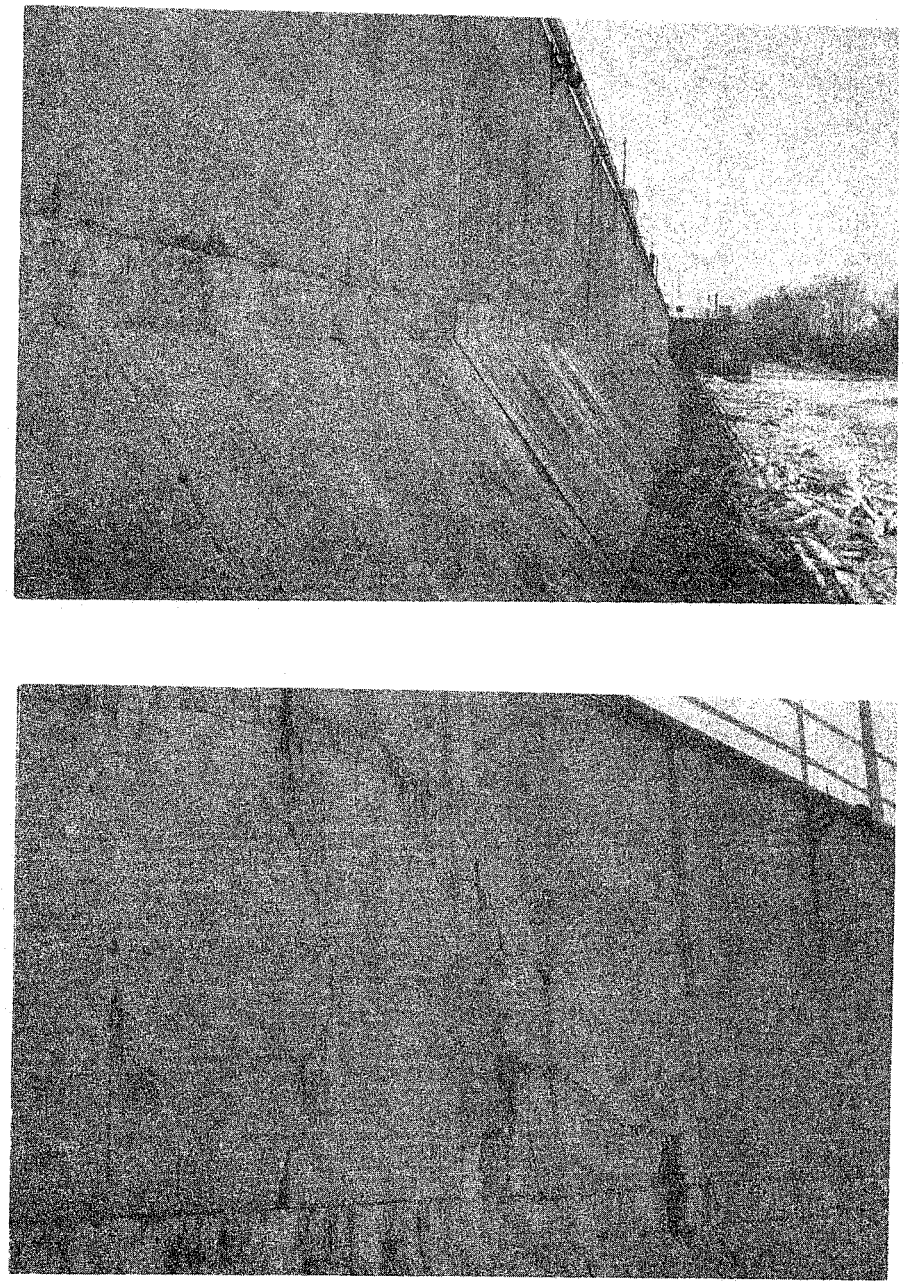


Figure 43. Typical condition of concrete overlay on back side of river wall, 1986, Dresden Island Lock

about 24 miles upstream from Pittsburgh, Pennsylvania, where the confluence of the Monongahela and Allegheny Rivers form the Ohio River, and about 105 miles below the navigable upstream limit of the Monongahela River at Fairmont, West Virginia.

Project history

88. The original locks and dam were constructed between 1905 and 1907. The locks were constructed by the Dravo Contracting Company of Pittsburgh, Pennsylvania, and the dam by the US Army Engineer Office, Pittsburgh, Pennsylvania, using hired labor. The structure has been operated and maintained since 20 May 1907. The construction history is summarized in Figure 44.

89. The original lock structure consisted of two 360-ft lock chambers situated on the right bank of the river. The landward lock chamber was provided with gate recesses and miter sill at midlength that would reduce the chamber length to 180 ft. These provisions were incorporated for low water lockages if necessary. Control of the filling and emptying systems was by 8-ft-diam cylindrical valves of the air-operated vertical lift type. The land chamber was filled through an intake in the land wall gate recess discharging through six ports in the upper miter gate sill, through an intake in the river wall upstream of the upper miter gate sill, and through an intake in the river wall through five ports in the middle wall. The chamber was emptied through valves in the gate monoliths of the land and middle walls discharging into the river downstream of the lower miter gates. The river chamber was filled through a valve in the river wall gate monolith discharging through six ports in the gate sill and through an intake in the river face of the river wall downstream of the upper gate, discharging through five ports in the river wall. The chamber was emptied through two valves in the river wall upstream of the lower gate, discharging into the river through the river face and downstream end of the river wall.

90. The original dam provided a gated structure with air-operated reverse sector type gates. The crest of the concrete section was at elevation 723.9 with the gate providing pool elevation 726.7. The concrete structure is supported on timber piles spaced on 4-ft 2-in. centers transversely and 4-ft centers longitudinally. No records exist defining the type and size of the bearing piles. It is speculated that the piles are 12-in. oak, consistent with normal practice at that time and with piles used in similar structures. Wakefield piling provides an upstream cutoff with a row of timber

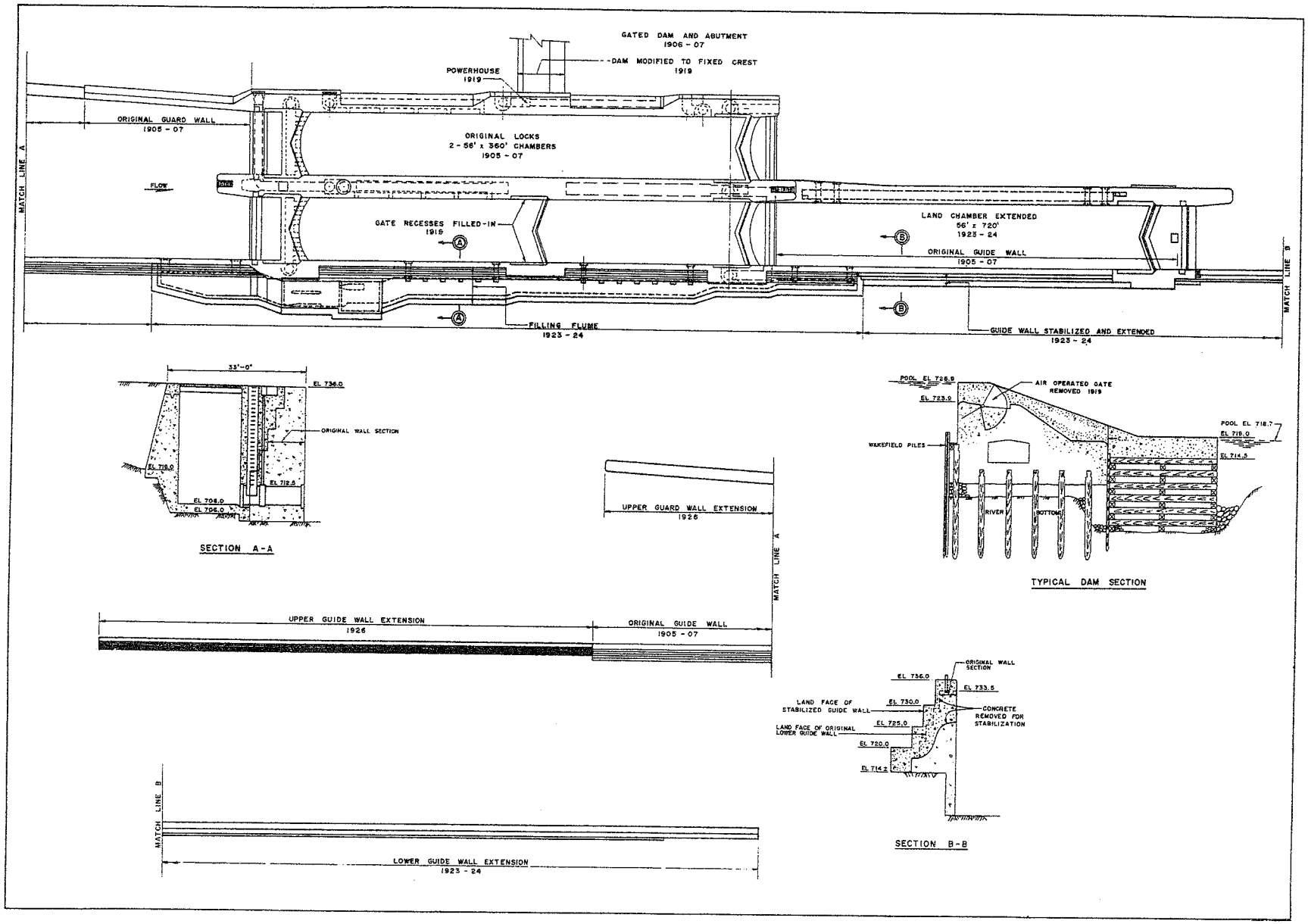


Figure 44. Construction history, Lock and Dam No. 3, Monongahela River

sheets driven at the downstream end of the section. An 18-ft-wide rock-filled timber crib was constructed downstream of the dam.

91. In 1919, a power house was constructed on the river wall by building an additional concrete monolith over the dam section adjacent to the river wall. The extra miter gate recesses at midlength in the land chamber providing for low water lockage were filled in with concrete at this time. Also in 1919, after the desired effect of the gated dam was not achieved, the gates were removed and a concrete section was added to the original concrete and stone-filled crib to create a fixed crest dam with crest elevation 726.9.

92. Major modifications, which included lengthening the land chamber to 720 ft and revising its filling and emptying system (Figures 45 and 46), were made to the lock structure during 1923 and 1924. The lock chamber was lengthened by extending the middle wall, land wall, and lower guide wall. Revision of the filling system consisted of constructing a flumeway adjacent to and behind the existing land wall. Five intake ports located in the land wall upstream of the needle dam sill filled the chamber through seventeen 54-in.-diam ports in the land wall face. Seventeen discharge ports of the same diameter were provided in the middle wall extension. Control of the filling and emptying ports was achieved with a hydraulically operated, air-powered, 54-in.-diam butterfly type valve in each port.

93. In 1926, the upper guide and guard walls were extended. The upper guard wall consisted of a concrete cap supported on a rock-filled crib. The upper guide wall monoliths were founded on rock.

94. By the 1970's, deterioration of the concrete and operating features was accelerating, necessitating more frequent repairs. The chamber faces of the locks had been repaired twice and were in need of repair again. The gates and related features had been repaired a number of times and the gates replaced several times. Budget cuts had just about eliminated "normal maintenance" and reduced repairs to those labeled "emergency." The repairs that were made tended to be more extensive, more costly, and closed the lock chamber for longer periods of time. A summary of the large repair items since 1930 is presented as follows:

Repair History

Date	Repair	Cost, $	Days Lock Closed
1919	Fill land lock gate recesses	2,300	
1930	Land lock; repair sills and replace upper gates	Not given	15
1931	River chamber; valves and sills	Not given	15
1932	River chamber; valves and gates	8,574	16
1932	Replace turbine and valve stems	23,781	
1935	Reface land lock chamber walls	29,936	Extended
1936	River chamber; repair sills and replace lower lock gates	7,130	
1936	Land chamber; valve, lock walls, and miter sills	12,951	22
1938	Land chamber; emptying valves	6,467	15
1939	Repair apron of dam	4,742*	
1939	River chamber, repair valves, upper gate and sills	13,829	
1940	Gunite river wall	11,072*	Extended
1940	Land chamber; repair miter sills and replace gates	15,971	
1943	River chamber; replace upper lock gates	7,846*	
1944	River chamber; repair miter sills, quoin seals, and valves	20,612	
1944	Land chamber; repair valves, gates, and miscellaneous repairs	63,760	37
1946	Repair top of lower guide wall	4,145	
1949	Land chamber; replace lower gates	13,981*	
1949	River chamber; repair sills and valves, replace gates	55,156	
1951	Repair gate anchorage and gate operating machinery	17,365*	14
1951	Repair upper middle wall gate	1,500*	
1952	Reface land chamber by guniting	19,981*	Extended
1952	Land chamber; replace upper lock gates	13,409*	
1952	Repair top of river wall	6,480*	
1952	Land chamber; repair miter sills, valve, and lower gate	27,951	13
1954	Reface upper guard wall and river chamber lock walls by gunite	17,310*	
1955	Land chamber; repair sills and valves	52,898	
1955	River chamber; repair sills, gates, and valves	44,880*	
1957	Spot reface river face of river wall	14,795	
1958	Repair top of middle wall	66,064	

* Estimated cost (actual not available).

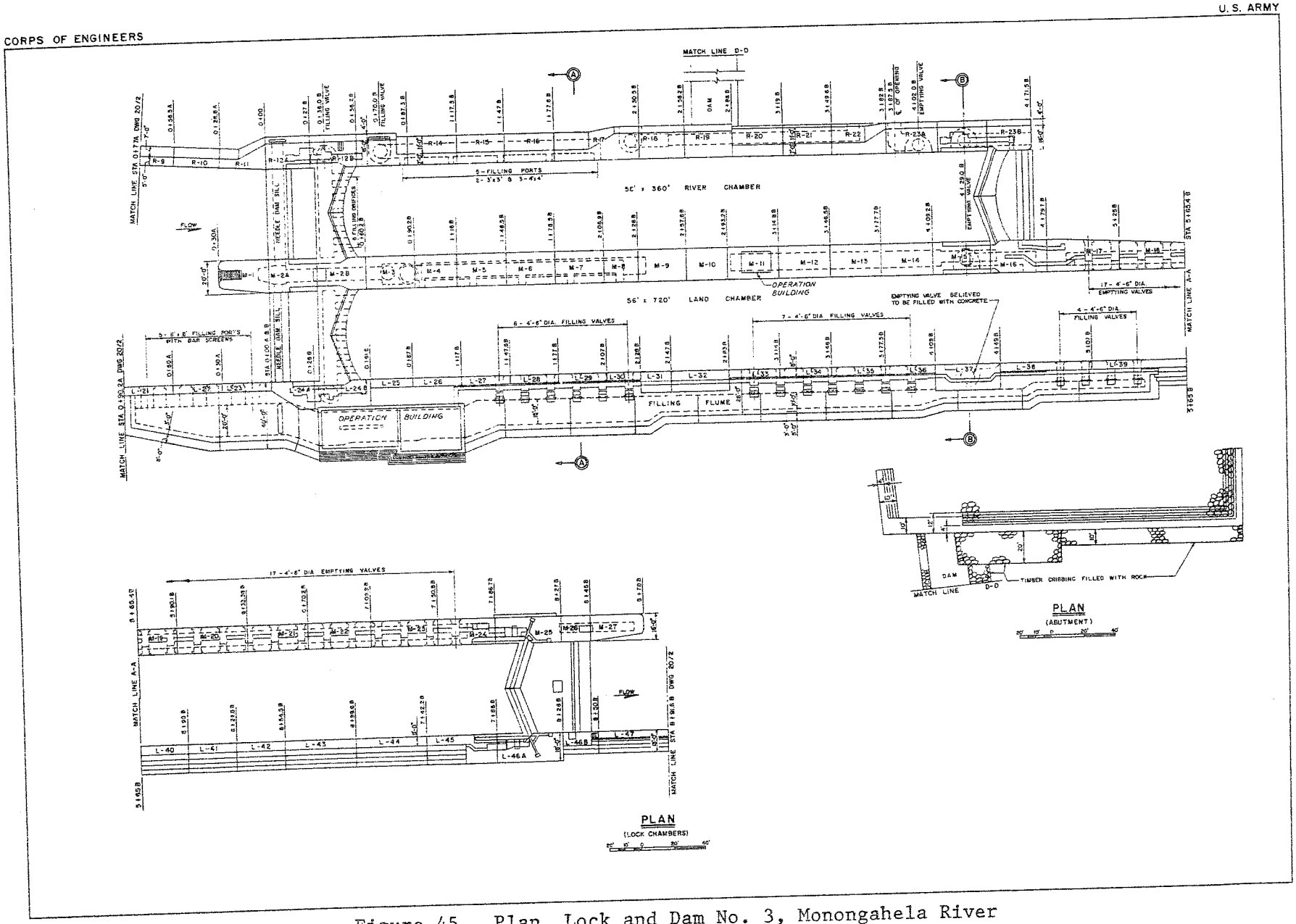


Figure 45. Plan, Lock and Dam No. 3, Monongahela River

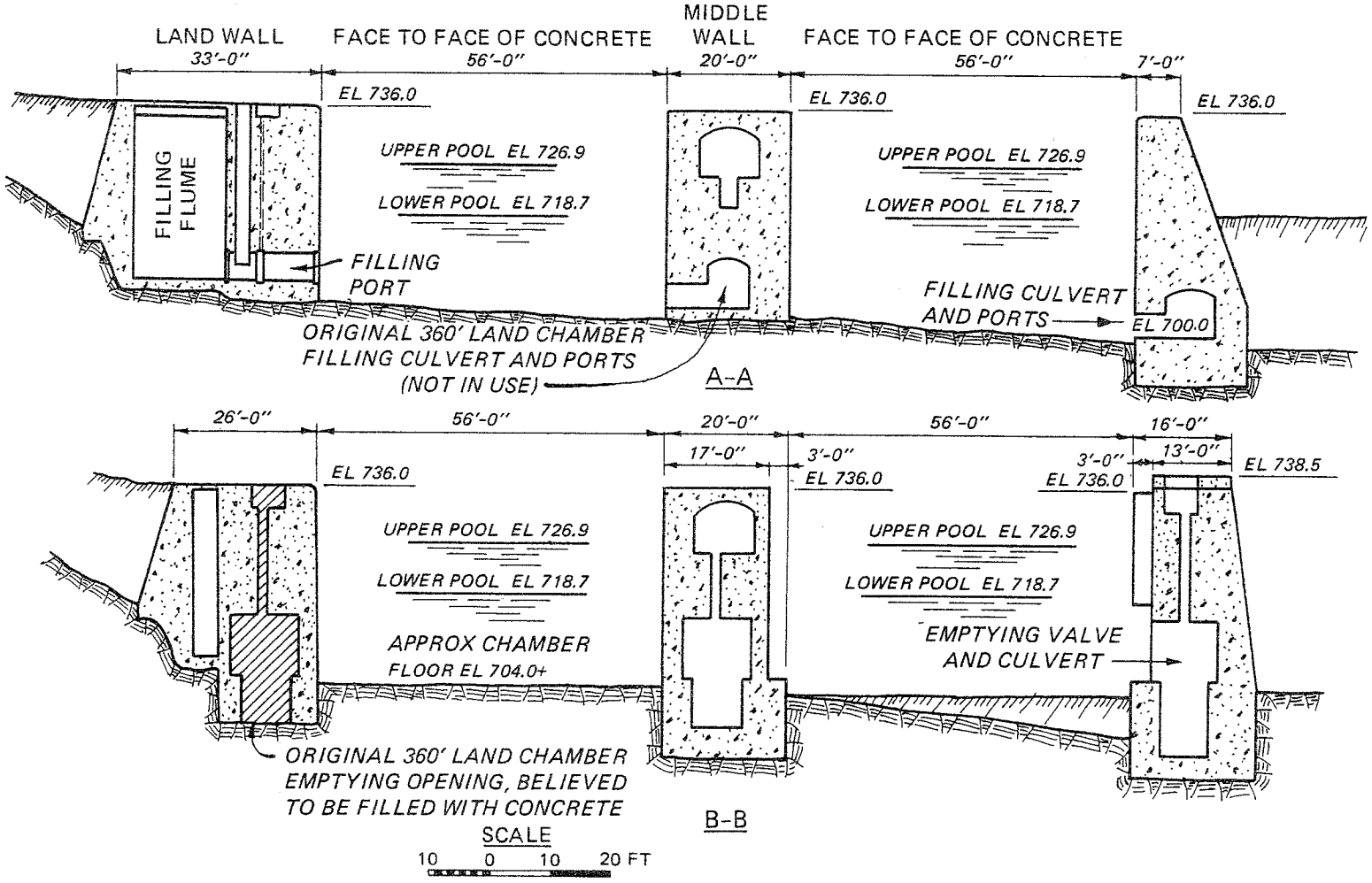


Figure 46. Typical sections, Lock No. 3, Monongahela River

Condition evaluation

95. The condition of the structure was evaluated by WES during the period October 1974 through June 1975 (Pace et al. 1976). The objective of the study was to conduct a detailed condition survey of the structure and operating systems along with an in-depth engineering evaluation and review of the condition, stability, and stress analysis of the lock masonry. As part of the preliminary work for this study, several related items were accomplished. In the Spring of 1973, the District conducted a crack survey which was supplemented later that year when WES performed a similar survey. Also in 1973, the District awarded a drilling contract for eight vertical 6-in.-diam cores through the concrete into the foundation rock, and 28 horizontal NX cores drilled into the wall faces. Ten additional vertical NX cores were later taken. These cores were used by WES in evaluating the condition of the concrete and foundation materials. The 6-in.-diam holes were also used for borehole photography.

96. From general observations, it was noted that the lock walls were badly spalled and scoured. Previous shotcrete repairs of the wall faces were almost completely deteriorated (Figure 47). The structural beams supporting

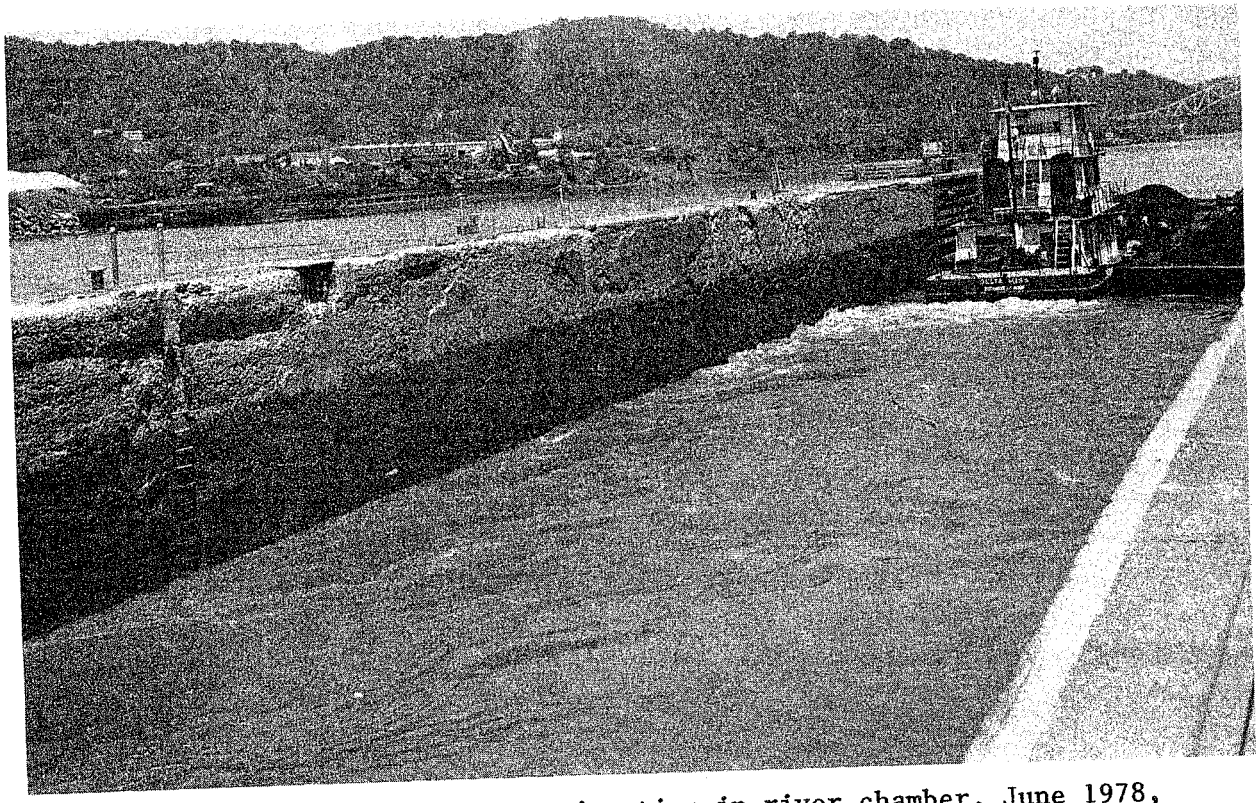


Figure 47. Concrete deterioration in river chamber, June 1978, Lock No. 3, Monongahela River

the slab over the filling flume were crushed at the ends and cracked with reinforcing steel exposed. Cracks were noted in the top of the slab and were found to extend through the slab. The concrete under the gate operating machinery was cracked and deteriorating.

97. The 6-in.-diam cores, the horizontal NX cores, and the borehole photography showed that the top 2 to 6 ft of concrete was badly weathered and deteriorated. The cores contained fairly close spaced parallel fractures characteristic of cyclic freezing and thawing action. There were several reasons, in addition to the age of the project, for the extent of concrete deterioration. At the time of the original lock construction (1905-1907) and the lock extension (1923-1924), the state of the art was such that the required quality control on items such as aggregate type and origin, cement quality, concrete mixture designs, placing and curing requirements, source of water, strength requirements, and testing procedures was much less restrictive than presently required. The lack of quality control and the absence of protective wall armor, steel reinforcement, and air entrainment admixtures made the lock walls susceptible to extensive abrasive and erosive action over the years. The high acidity of the river water resulting from mine acid drainage further aggravated the condition by destroying the cementitious material in the surface concrete. The wear and aging process was accelerated by all these conditions and deficiencies.

98. The foundation drilling disclosed several zones of possible weakness in the foundation rock. The foundation rocks recovered were essentially flatlying, cyclic sediments consisting of shale, limestone, siltstone, and undurated clay. Coal is present in minor quantities. Cores taken in the gate monoliths indicated weathered and badly fractured rock for an average depth of 8.5 ft. Borehole photo logs showed areas where there is poor contact between the concrete and foundation rock.

99. A stability analysis was made on selected monoliths of the locks, the dam, and the abutment to determine if adequate resistance against overturning and sliding and excessive base pressures existed. Only one monolith of each typical configuration and loading was analyzed. The results of the stability investigation showed the land wall monoliths did not meet the required values for percent active base and shear-friction factor of safety. In most cases, the values were significantly below the allowable with the position of the resultant for several of the monoliths falling outside the base.

The land wall monoliths were also checked using active earth pressures with the resulting values still being below the allowable values. Foundation pressures were considered excessive for the majority of the monoliths. The middle wall monoliths generally met the required minimum percent active base but were deficient in shear-friction factor of safety. The river wall monoliths also generally met the required active base, but there were several exceptions. The shear-friction factors of safety were lower than the allowable with several of the allowable base pressures exceeded.

100. For the stability analysis of the fixed crest dam the normal operating condition was analyzed, assuming normal upper and lower pool levels, and the results were within acceptable limits. Allowable pile loadings were assumed based on prior experience with similar structures (horizontal, 6 kips/pile; compression, 48 kips/pile). The actual loadings were within those values. The dam abutment was analyzed using the assumed allowable pile loads for the fixed crest dam analysis. The position of the resultant fell outside the base of the abutment and caused excessive tensile and compressive stresses in the supporting timber piles. These stresses account for the riverward cant of the abutment wall.

101. In summary, the investigation identified five areas as critical and in need of emergency repairs. These areas were the upper guide wall, the upper guard wall extension, the filling flume retaining wall, the lower gate monoliths on the middle wall, and the lower guide wall end monolith.

Rehabilitation

102. When it became apparent that major repairs would be required to assure the continuation of the locking capability of Lock No. 3, a rehabilitation plan was formulated (Pittsburgh District 1976) and carried out which included replacing the upper guard wall extension, anchoring unstable walls, renovating gate and valve operating machinery, refacing and resurfacing lock walls, changing the operating system from air to hydraulic, and renewing the electrical system. Replacing the upper guard wall extension and anchoring the land chamber filling flume wall were considered critical items and were accomplished under separate contracts in 1977. The remaining work was included under a contract awarded in June 1978 to the Dravo Corporation of Pittsburgh, Pennsylvania.

103. Several plans for accomplishing the major rehabilitation work while minimizing the impact on the local economy were studied. It was

determined that the most practical and least economically damaging alternative was to close the chambers one at a time and extend the river chamber from 360 to 720 ft to adequately pass traffic while the land chamber was closed (Figure 48). Under this plan, the work was carried out in two phases (Pittsburgh District 1980a). The first phase covered all work in the river chamber, including extending it by constructing a sheet-pile cell river wall extension, a lower gate monolith, and miter sill. The contractor also performed work on the land chamber which did not interfere with locking during the first phase. The second phase included all work on the land chamber which necessitated the closure of the chamber.

104. The upper guide wall was constructed during the period 1905-1907 and extended in 1926. Severe scouring and gouging had occurred at areas where tows had hit the wall in lining up for lock entry. Wall faces had been scoured back an average of 6 to 8 in. Maximum gouge depth in one reach of the wall was 36 in. (Figure 49). Wall thickness in the area of the gouging is 4 ft from the top of the wall to elevation 731.83 and 5 ft 2 in. between elevations 731.83 and 728.17. Continued scouring and gouging could have resulted in failure of the guide wall. One section from Station 4+25A to Station 6+52A was designated as extremely critical, and the repairs were started by District hired-labor forces in August 1976.

105. The work was done on one monolith at a time beginning at Station 4+25A. Old concrete was removed from the face of the wall by jackhammer where necessary to provide a minimum depth of 12 in. for the new concrete. The repair extended from the top of the wall to the waterline (approximately 6 ft). After the concrete was removed, No. 7 dowels were grouted into the wall on 2-ft centers. Initially Sikadur Hi-Mod Gel, as manufactured by the Sika Corporation, was used to anchor the dowels. The gel proved difficult to inject into the holes, so a sand-cement grout was used instead. Wire mesh, 6 by 6, No. 6, was attached to the dowels. Three 35-ft sections of used 90-lb railroad rail were positioned in the wall, so the rail would extend 1/2 in. past the face of the wall. The rails were spaced 1 ft 8 in. apart vertically to act as wall armor (Figure 50). When the reinforcing and rails were in place, forms were erected for the wall face and concrete was placed. Ready-mix concrete with Type II cement was used. By mid-December, the work had been completed to Station 5+87A. High water and adverse weather conditions made it impractical to continue. The rest of the upper guide wall was repaired by the

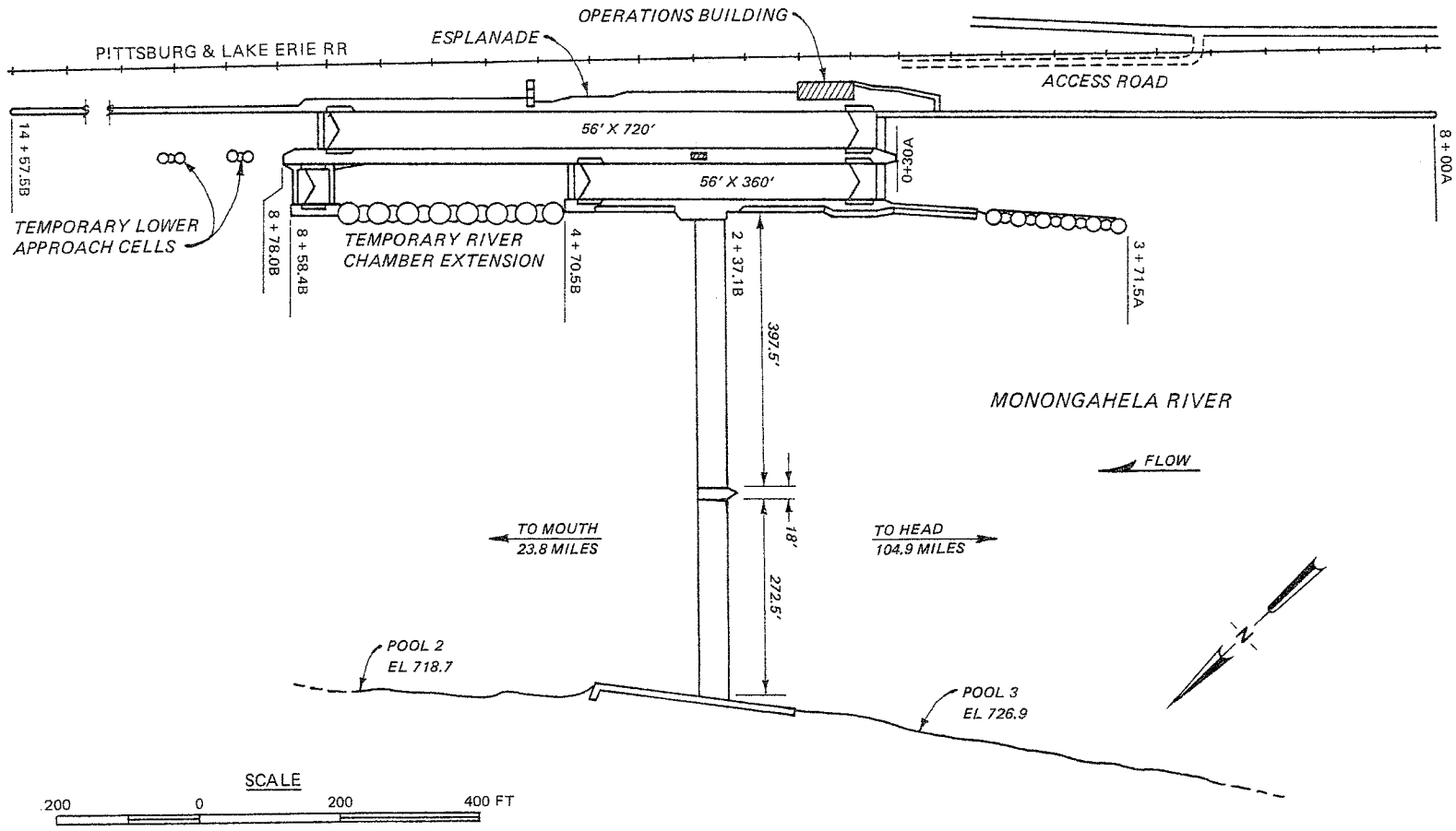


Figure 48. River chamber extension, Lock No. 3, Monongahela River

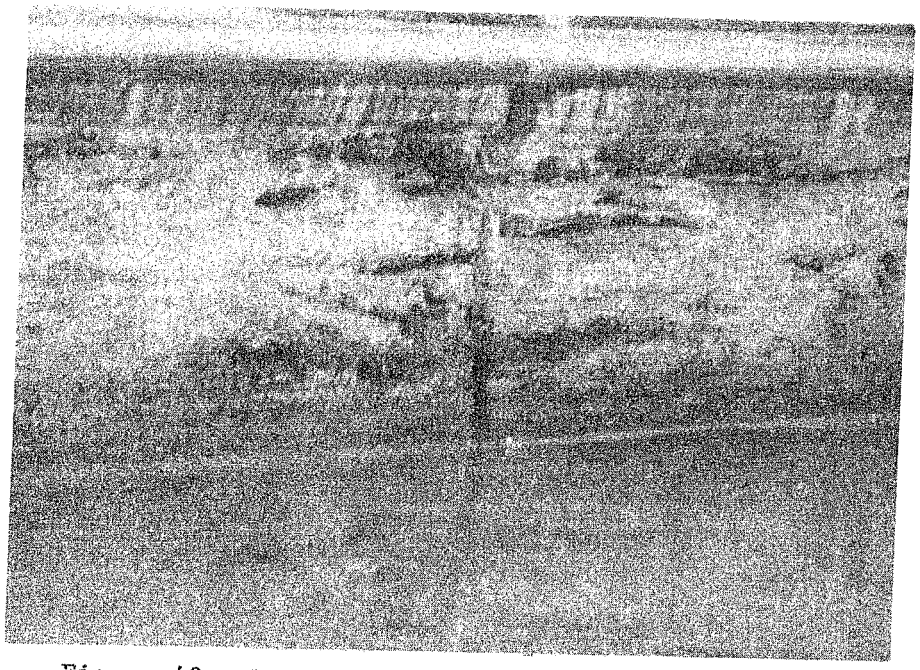


Figure 49. Upper guide wall deterioration, 1974, Lock No. 3, Monongahela River

Dravo Corporation in 1980 under the major rehabilitation program. The total cost of the hired-labor work was $89,600.

106. The 200-ft-long upper guard wall extension was constructed in 1926. It consisted of a concrete cap on rock-filled cribbing. Investigation in 1975 revealed the entire guard wall extension had settled approximately 9 in. and was canted riverward about 5 or 6 in. The wall was visibly out of alignment. A diver's inspection revealed that the top timbers of the wider bottom section upon which the top section and concrete cap rested were broken. Although all the timbers in this area could not be inspected because of silt and debris, it was suspected they had also failed and caused the riverward cant. The stability analysis confirmed this area as a problem area because of excessive bearing pressures at the plane between the upper and lower cribs. It was concluded that tow impact, along with deterioration of the timbers and potential loss of crib fill, would ultimately have caused the complete collapse of the wall.

107. The remedial solutions considered included bracing on the riverward side by means of sheetpiling or a rock berm. These solutions were rejected since they would not have deterred the vertical settlement. The sheet-pile bracing by itself would not have provided any appreciable lateral

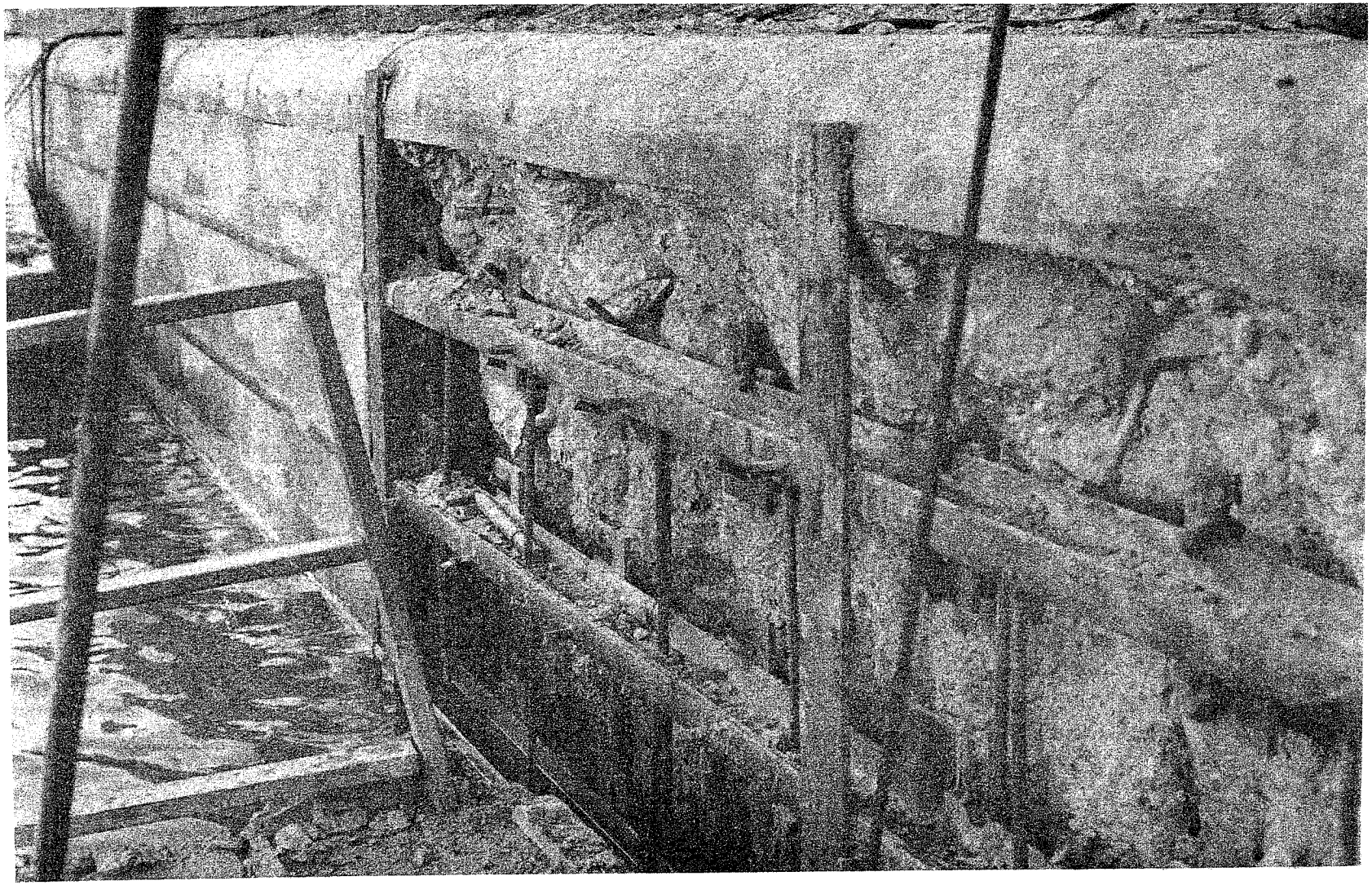


Figure 50. Upper guide wall repairs, December 1976, Lock No. 3, Monongahela River

support. A rather substantial rock berm would have been necessary to be effective. Neither of these, or other support measures, would have remedied the deteriorated condition of the concrete cap and cribbing. A decision was made to remove the entire wall and replace it with a new guard wall extension of steel sheet-pile cell construction. A contract for this work was awarded to Crain Brothers, Incorporated, Sewickley, Pennsylvania, in January 1977.

108. A derrick-boat was used to remove the existing wall. The concrete cap was broken with a headache ball and then loaded into barges with a clamshell bucket for disposal. The timber cribbing and rock fill were also removed with a clamshell bucket. The river bottom was cleaned and graded in preparation for sheet-pile placement.

109. The new guard wall extension consists of six circular sheet-pile cells, 24.4 ft in diameter, connected by five arcs. The downstream most cell, No. 6, was not attached to the upper guard wall. Its downstream side is located approximately 12 ft from the wall to maintain an opening to improve approach conditions during high flows. The total length of the wall is 208.7 ft. Government-furnished PS-28 steel sheet piling was used for the cells. All sheet piles were driven to rock. Prior to driving, the portion of the piling which is 2 ft above and 2 ft below normal upper pool was sandblasted and painted with coal tar epoxy.

110. The upstream-most cell, No. 1, was filled to elevation 729.0, 2.1 ft above normal upper pool, with tremie concrete, then later filled to elevation 736.0 to make a totally concrete-filled cell. The tremie concrete was a 7.0-bag/cu yd mixture, with a 7-in. slump and 4.2 percent air entrainment. Type IS cement was used. All the contractor's concrete was ready-mixed concrete delivered by truck from a local supplier. The concrete was transferred to 4-cu yd buckets on barges at a dock 0.4 miles from the jobsite. In order to maintain a continuous pour of tremie concrete, two towboats with barges and buckets were used. At the cell, the tremie concrete was transferred to a hopper on top of an 8-in.-diam tremie pipe.

111. All other cells and arcs were filled with clean, free-draining sand and gravel and capped with 2 ft of concrete. The sand and gravel was dredged from the Monongahela River immediately below Lock and Dam No. 2. This material had previously been used for cell fill with good results. The concrete for the caps was a 5.5-bag/cu yd mixture, with an 8-in. slump and 4.0 percent air entrainment. The caps are reinforced by two layers of 6- by

6-in. W4 by W4 wire mesh. A curing compound, Horncure 30D, manufactured by W. R. Grace and Company, was applied to the surface. The concrete was also covered with light burlap to protect it from the sun.

112. A steel fender system, consisting of four rows of TS 10 by 10 tubing filled with concrete, was constructed on the landside of the cells. The upper guard wall extension (Figure 51) was completed in September 1977 at a

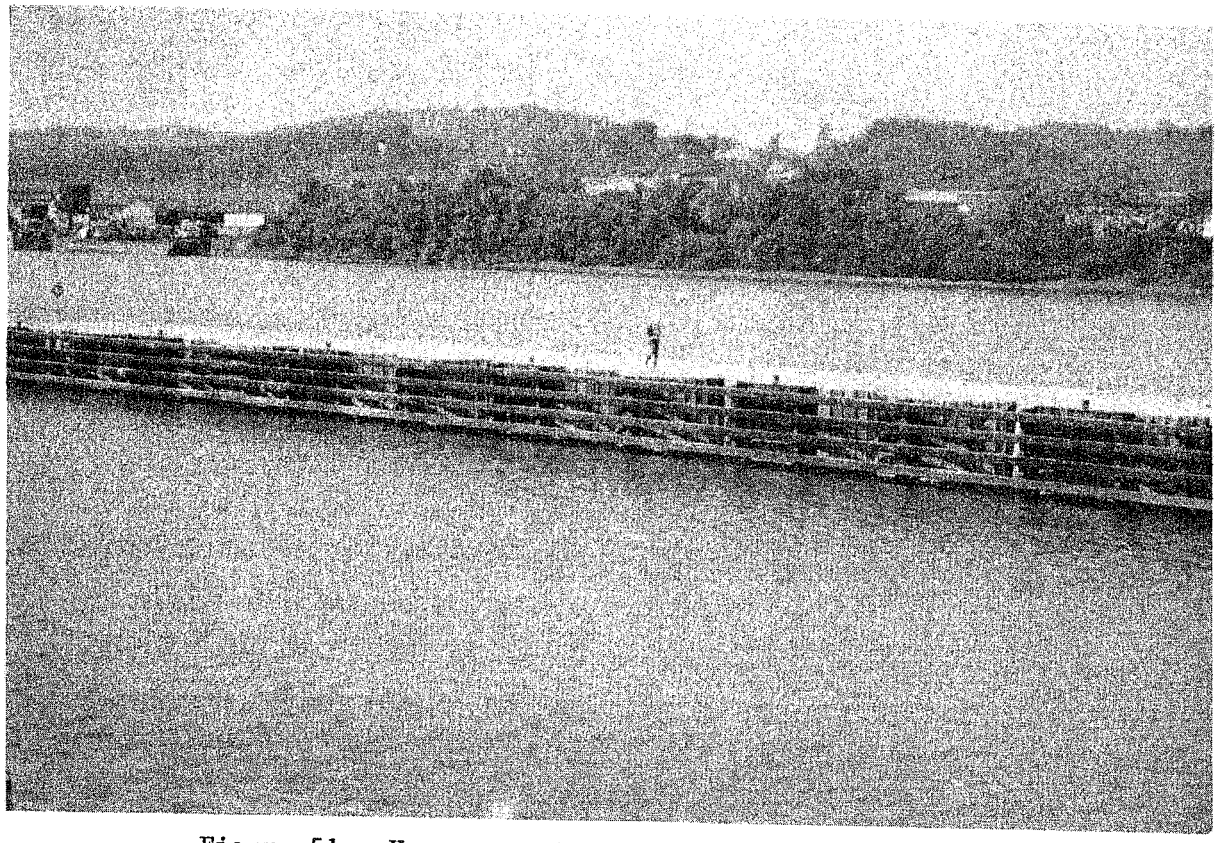


Figure 51. Upper guard wall extension, October 1977, Lock No. 3, Monongahela River

total contract price of $520,448. The contractor, Crain Brothers, Incorporated, had a lot of prior experience with cell construction and thus was instrumental in successfully completing the work on schedule. There was no settlement in any of the concrete caps, and the caps were free of cracks. The uppermost cell, which is concrete-filled, had a few hairline cracks in the top surface.

113. The filling flume for the land chamber was constructed in 1924. The landward retaining wall was found to be inherently unstable, relying on the flume deck bearing against the land wall to support it. The deck is of concrete beam and slab construction. Structural cracks extended through the

slabs and beams. The beams were crushed at both ends, and the deteriorated concrete in the beams exposed the reinforcing steel. There was evidence that some movement of the flume wall had occurred. With continued deterioration of the slab and beams, failure of the wall would have occurred.

114. It was determined that the best method of stabilizing the flume wall was with rock anchors (Figure 52). The original design called for the

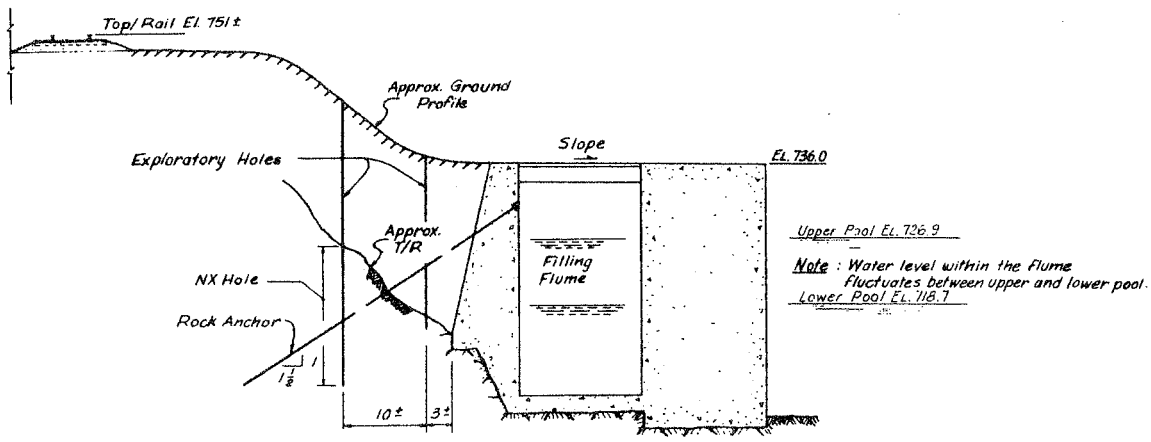


Figure 52. Section through land chamber filling flume looking upstream, Lock No. 3, Monongahela River

installation of 97 rock anchors on an average spacing of 6 ft. Tendons were to be 1-1/4-in.-diam high tensile steel Dywidag thread bar anchors. Four of the anchors were to be tested to 140 kips or 75 percent of the ultimate bar strength, and all the bars were to be locked off at 112.5 kips or 60 percent of the bar strength. The design procedure was to drill a 4-in.-diam hole through the concrete wall and a 3-1/2-in.-diam hole through the overburden. Then a 3-in.-diam casing was to be installed in the hole. The hole into rock was 3 in. in diameter. The specifications called for testing for watertightness and pregrouting if necessary with subsequent redrilling. The bond length of the bars in rock, 20 ft, was to be grouted with a cement grout, and after a minimum setting time of 7 days, the bar was to be stressed. After stressing and checking, the stressing length of the bar would be grouted for corrosion protection.

115. In May 1977, a contract for the installation of the rock anchors was awarded to Nicholson Anchorage Company of Bridgeville, Pennsylvania. The work commenced on 3 June 1977. Nicholson proposed, and the Corps approved, several modifications to the specified materials and procedures. The tendon used was a 1-3/8-in.-diam "Stressbond" bar manufactured by Stressteel

Corporation of Wilkes-Barre, Pennsylvania, from 160-ksi minimum steel. The ultimate strength of this bar is 237.6 kips. The spacing was increased to an average of 8 ft, and the total number of anchors was reduced to 72. The testing requirement was modified so that the first 10 anchors installed, and thereafter one every monolith or not less than one out of five be tested to 190 kips, 80 percent of the ultimate strength of the bar. The lock-off load, 60 percent ultimate, was 142.8 kips.

116. Anchor boreholes, 4-1/2 in. in diameter, were drilled on a 1.5V on 1H slope through the concrete and the overburden. A standard 4-in.-diam Schedule 40 pipe was placed through the overburden and 1 ft into rock. A 3-1/2-in.-diam hole was then drilled 20 ft into rock. The drilling equipment used by the contractor consisted of a crawler-mounted hydraulic drill rig specially designed and built for anchor installation (Figure 53). Because of the

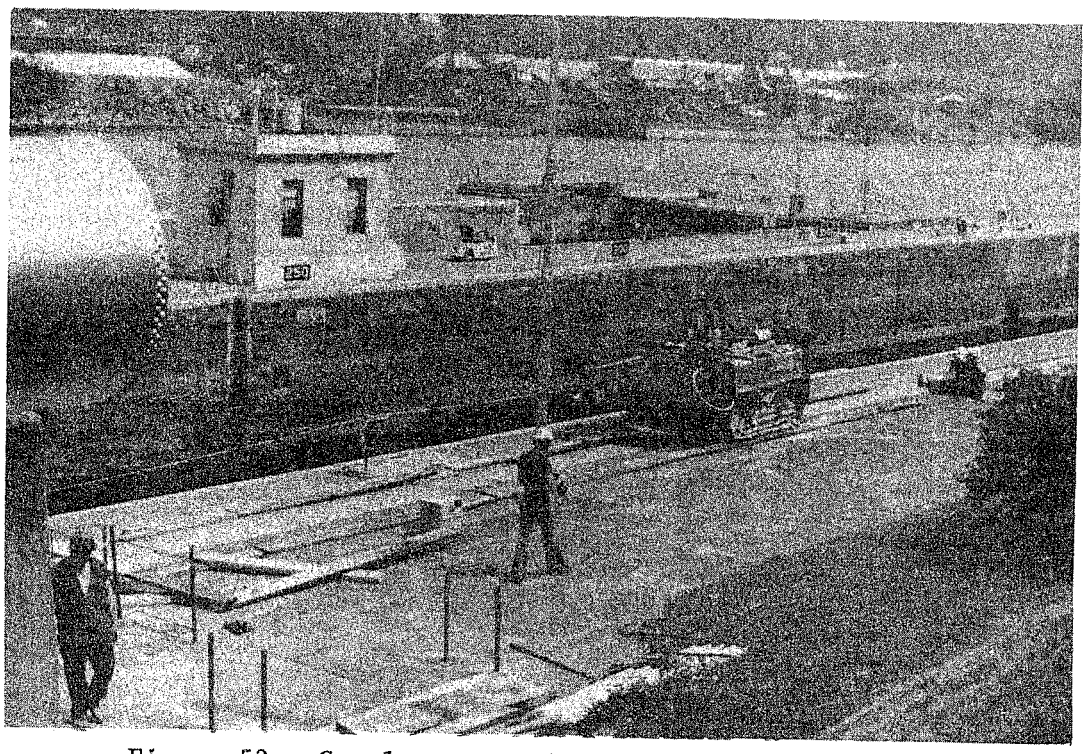


Figure 53. Crawler-mounted drill rig used for anchor installation, Lock No. 3, Monongahela River

limited load-carrying capacity of the concrete slab over the flume, the contractor was required to place planking across the beams spanning the deteriorated slab. A high-pressure water flush was used to remove drill cuttings and clean the boreholes.

117. Installation of the filling flume anchors was especially difficult

because downstream of the operations building the anchor heads are located 5 ft below the top of the filling flume slab (Figure 54). Another difficulty was that high water is fairly common at Lock No. 3, and the contractor had to install scaffolding or work platforms inside the flume that could be easily raised. The contractor had to break an opening in the slab and drill at a 1V on 1.5H slope through the concrete retaining wall and the overburden material while advancing a 4-in. diam casing and, finally, into the rock for a depth of at least 20 ft. The slope and the vertical location of the anchors were optimized by many trials using various slopes and locations. The controlling criterion for the design of the anchors was to keep the resisting-to-overturning moment ratio around 1.3. This value is not usually computed for new structures. In the Pittsburgh District when investigating stability for existing structures undergoing rehabilitation, it is found that this ratio gives a better perspective of possible overturning failure, more than just the percent active base criteria (Krysa 1982).

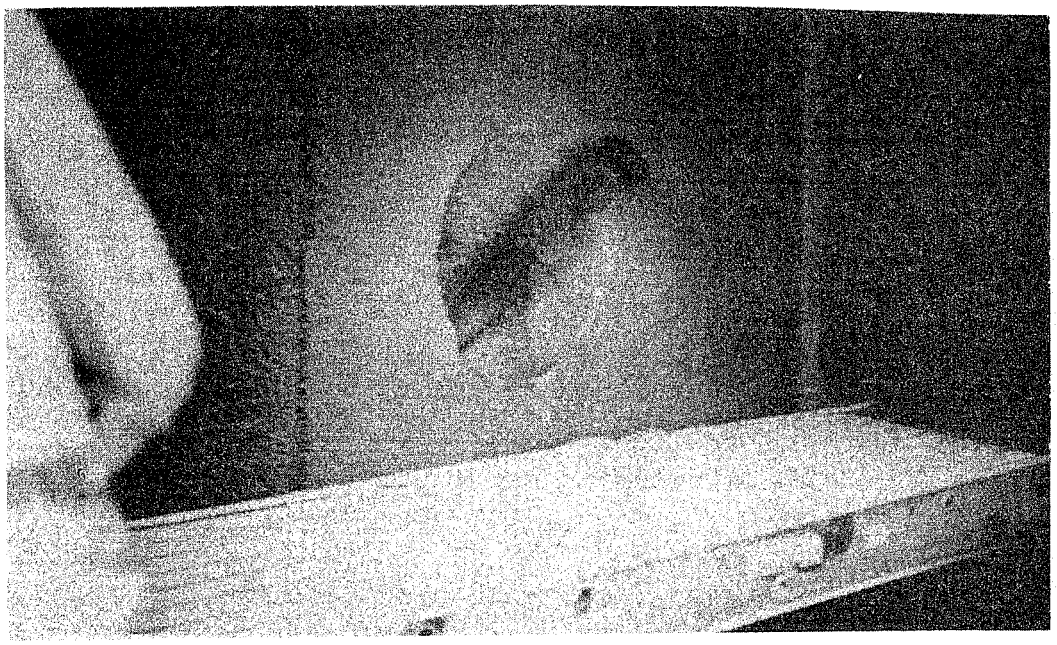


Figure 54. Installation of filling flume anchors, Lock No. 3, Monongahela River

118. The requirements for testing watertightness and pregrouting were eliminated. Instead the contractor adopted a one-stage grouting process which assured all crevices and fissures were filled. Grout was pumped into the borehole through a 2-in. grout pipe. Pumping was continued until fresh grout overtopped the 4-in. casing at the surface. The grout level was monitored for

a period to see whether there was any loss into voids or fissures in the rock. In only two cases was there any grout loss, and this loss was caused by small interconnecting fissures between boreholes. In these cases, grouting was done in a manner which filled the interconnected holes at the same time.

119. The grout used was Atlas Type III cement mixed in a high speed colloidal mixer with water at the rate of 5 gal per 94-lb bag of cement. No admixtures were used. An expansive agent had been specified, but the contractor requested a variance on technical grounds. He contended that based on his experience, expansive grouts worked effectively only when fully constrained. In an anchor borehole, full constraint is not possible, and thus the grout is free to expand without restraint up the borehole. This loss of density is accompanied by a loss of strength. Since the grout is a main structural member in the load-carrying system, this expansion was undesirable. The variance was allowed.

120. After it was established that the grout level was stable, the anchor bar was inserted to full depth using a cherry picker to handle the bars. The stressing length of the bar was coated with No-Ox-Id Type A grease and encased with polyethylene tubing. This coating was done for debonding purposes and corrosion protection. By testing grout cubes, it was determined that the grout was not attaining the specified strength of 4,000 psi in 7 days. Therefore, the contractor waited 14 days before stressing and testing the anchors. A standard Stressteel arrangement of a 7- by 7- by 2-in. wedge plate and three-piece wedge set was used to lock off the required load. Stress was applied by means of a hollow center 100-ton capacity jack that was situated on top of a second smaller jack which was used to set the lock-off wedges. The anchors were stressed to 142.8 kips, and the tendon elongation was recorded at 20 percent, increments of the design load. The anchors were then further stressed until approximately 1/4-in. additional extension was achieved, and the wedges were then set hydraulically with the small seating jack. The extra extension was to allow for losses during seating of the lock-off wedges. Immediately after lock-off, a lift-off test was performed to determine the actual stress in the bar. Where losses had been too great during wedge seating, the lock-off operation was performed again. A minimum of 1 hr after the first lift-off test, a second lift-off was performed. The second lift-off load was required to be within 3 percent of the first. All anchors achieved this requirement.

121. Testing was carried out by loading the bars in 10-kip increments up to 190 kips, 80 percent of ultimate, and recording the elongation at each load level. The maximum load was held for 2 hr. After any load loss was noted, the anchor load was reduced to zero with elongation recorded at 100 kips, 50 kips, and zero. All test anchors performed satisfactorily and upon completion of tensioning, the anchorage recesses were filled to the original concrete surface with nonshrink mortar.

122. The design and installation procedures described resulted in a final product of high quality and reliability. The fact that none of the 72 rock anchors was rejected for failing to meet performance standards supports that conclusion. The contractor's experience with installing rock anchors contributed significantly to the efficient and timely progress of the work. When used to drill long sloping holes, the drill rod may have a tendency to deflect downward causing problems later when the casing and the anchor bars are installed. For the drill rod, the contractor used 3-in.-diam steel pipe which had sufficient strength to minimize deflection.

123. The contractor's proposed elimination of testing for watertightness, pregrouting, and redrilling resulted in a savings to the Government of approximately $28,000. From the load test results, it can be concluded that the modifications had no adverse effects on the performance of the anchor. The only problem encountered during the work was unanticipated sheetpiling behind the flume wall which affected anchors 15 through 19. Apparently, the sheetpiling was used in the original construction and left in place. The drill bits would not penetrate the piling. Lance bars were used to burn through the steel.

124. Emergency remedial work on monoliths M-16, M-24, and L-61 was included in the flume wall anchor contract. Major cracks existed in middle wall gate monoliths M-16 and M-24. These cracks were allowing interior concrete deterioration through water percolation. The crack in M-24 was so extensive that a large block of concrete was isolated from the rest of the monolith. Continued deterioration could have caused failure of the walls. The end monolith of the lower guide wall, L-61, had been stuck by a tow, and a large section (approximately 60 cu yd) had been cracked and separated from the rest of the wall. Although the block was still in position, there was concern that it could be dislodged by further impact.

125. Monolith M-16 was repaired by the installation of three vertical

rock anchors through the cracked area to tie the concrete together. The surface concrete was removed, and the grout base was placed for the anchor bearing plates. The same procedures that were used for the flume anchors were used to install 1-in.-diam "Stressbond" bars. These bars were stressed to 10 kips. Cracked, deteriorated, and spalled concrete was patched with quick-setting grout. Monolith M-24 was repaired by drilling twelve 2-1/2-in.-diam holes through the cracked area into sound concrete and grouting in No. 8 dowels. The spalled and deteriorated concrete was patched with quick-setting grout.

126. The broken section of monolith L-61 was reinforced by placing a concrete block on the landside of the wall. The reinforced concrete block is 31 ft long by 4 ft wide by 7 ft high. It was anchored to the monolith and the broken section with No. 8 dowels on 2-ft centers. The crack was filled with a cement-sand grout. All work under the flume wall anchor contract was completed in October 1977 at a total cost of $234,304.

127. The stability analysis performed in conjunction with the condition survey and structural investigation identified all the lock wall monoliths except a few on the middle and river walls as unstable under the criteria used for evaluation. The monoliths were deficient in either percent active base or sliding factor of safety. The criteria require that the resultant of all forces fall within the middle third of the base for the normal operating condition. For maintenance conditions, the resultant may fall outside the middle third of the base provided that a minimum of 75 percent of the base area is in compression. For sliding, the shear-friction factor of safety should be at least 4.0 for normal operating conditions and 2.67 for maintenance conditions. Increasing the stability of the existing walls to meet the criteria would have been difficult, expensive, and in some cases impractical. The necessary spacing and resulting loads imposed by anchors could have possibly been detrimental to the walls. Since the performance of the monoliths, some 50 years old and some 70 years old, had been satisfactory, as evidenced by the lack of movement, the stability requirements were reduced. After reviewing the existing conditions, the following requirements were established:

	Percent Active Base	Sliding F.S.	Mr/Mo
Normal operating condition	90	2.2	1.50
Maintenance condition	70	1.8	1.30

128. A system of rock anchors was designed (Figure 55) to upgrade the stability of the lock walls to the reduced requirements. The original design called for one hundred eleven 1-1/4-in.-diam anchors in the upper guide wall, sixty-two 1-in.-diam anchors in the lower guide wall, twenty-three 1-in.-diam anchors in the land wall, one hundred twenty-eight 1-1/4-in.-diam anchors in the middle wall, and seventy-six 1-1/4-in.-diam anchors in the river wall. The anchors on the land and guide walls were to be locked-off at 20 percent of ultimate, 25 kips for the 1-in. bars, and 40 kips for the 1-1/4-in. bars. The anchors on the middle and river walls were to be locked off at 60 percent of ultimate, 112.5 kips.

129. All holes were to be drilled 4-1/4 in. in diameter for the full length through the concrete and the required bond length into rock (20 ft for the guide and land walls; 25 ft for the middle and river walls). The holes were then to be pressure-grouted from the bottom of the hole into the concrete a sufficient distance to seal off any cracks or crevices to ensure that the bonding grout would not be washed away from around the bar. The grouted section was to be redrilled, the anchor bar installed, and the anchorage length grouted. After a minimum of 7 days, the bar was to be stressed and locked off. An immediate lift-off would determine the actual stress in the bar, and a second lift-off, a minimum of 2 hr later, would determine if any losses had occurred. If the bar proved to be properly anchored, the stressing length would be grouted for corrosion protection. This procedure was intended for cement-grout anchors. After a polyester resin grout manufacturer assured the District that grout cartridges could be used successfully in this application, the option to use a polyester resin grout was added to the specifications.

130. The installation of the rock anchors in the lock walls was included in the contract awarded to the Dravo Corporation in June 1978. Dravo subcontracted the anchor work to Engineering Construction International (ECI), Incorporated, Pittsburgh, Pennsylvania. On 19 July 1978, ECI submitted its proposed procedure for installing the anchors. They chose the polyester resin grout option for anchoring the bars and cement grout for the secondary grouting. The anchors were Dywidag thread bar anchors, manufactured by Dickerhoff and Widmann, Incorporated. They intended to drill a 3-in.-diam hole for the full depth, pregrout it, and then redrill it at a 3-in. diameter to the bottom of the stressing zone and at 2-1/4-in. diameter for the bond length. The polyester resin cartridge manufacturer, Celtite, recommended the 2-1/4-in.

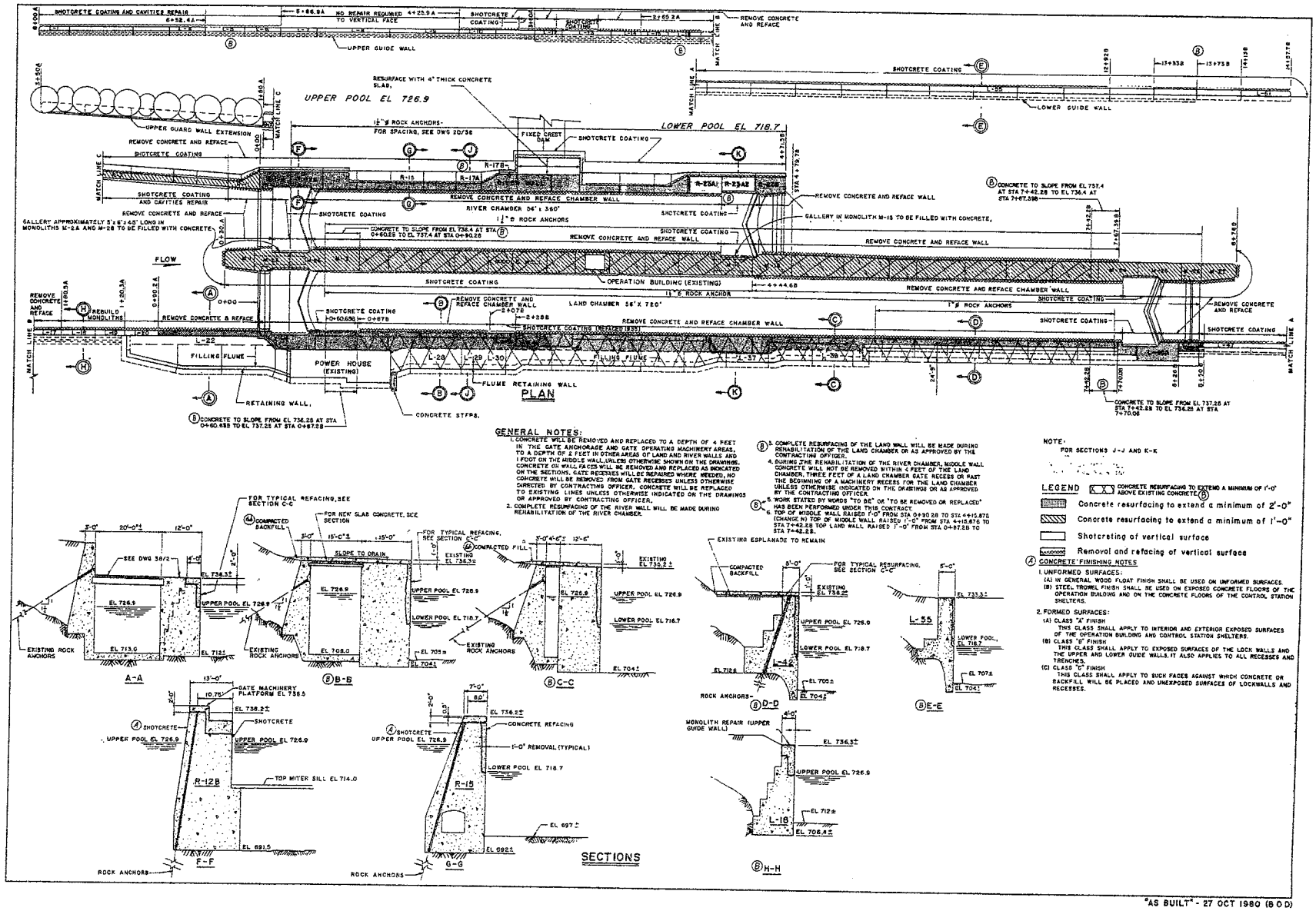


Figure 55. Refacing, resurfacing, and anchoring details, Lock No. 3, Monongahela River

hole for proper mixing of the 45-mm grout cartridge to be used.

131. According to the manufacturer, the polyester resin grout is capable of being mixed under water without affecting its bonding to the bar or the rock. For that reason, a determination was made to eliminate the costly and time-consuming pregrouting and redrilling. On 28 August 1978, the Corps issued to Dravo a directive eliminating the pregrouting and restoring the hole size in the stressing length to 4-1/2 in. for proper corrosion protection. ECI objected to the modification contending pregrouting was necessary for stabilizing the hole walls and for the proper performance of the resin grout system. It is believed that the real reason for the protest was that his bid was unbalanced and relied heavily on overruns in grout-take in the anchor holes (Krysa 1982). Prior core borings had shown that the rock was sound and would not adversely affect the performance of the resin grout.

132. The contractor began working as directed the week of 11 September 1978. He was initially using two Sullair Model 750 air track drills and rotary percussion hammers. When the work fell behind schedule, two additional rigs were placed in operation. Air was supplied by a Sullair compressor which delivered 1600 cfm at 100 psig. The drill bits used were a 4-1/2-in. button and a 2-1/4-in. four-blade chisel. The drill rods were approximately 1-1/16-in. and 1-5/8-in. O.D.

133. Shortly after beginning work on the middle wall anchors, ECI complained that the holes were caving and the anchor rods could not be installed. There were 15 middle wall anchors on the prime contractor's critical path. ECI claimed that increased progress could be achieved by pregrouting. In the interest of expediting the overall project, the Corps agreed to allow ECI to pregrout the first 15 holes, but stipulated an alternative solution should be found. The 15 anchors were installed (Figure 56), but with no improvement in the rate of work. The contractor continued to have drilling problems. At a meeting on 11 October 1978, the Corps directed ECI to install a 3-in.-diam casing in the 4-1/2-in.-diam hole prior to drilling the 2-1/4-in.-diam hole. The Corps had concluded that the material in the hole that the contractor claimed was from caving was actually the drill cuttings. The change in hole diameter from 2-1/4 to 4-1/2 in. was causing a pressure drop which allowed the cuttings to fall back into the hole. The drilling was being impaired by the cuttings. The 3-in. casing improved this condition, and there were fewer

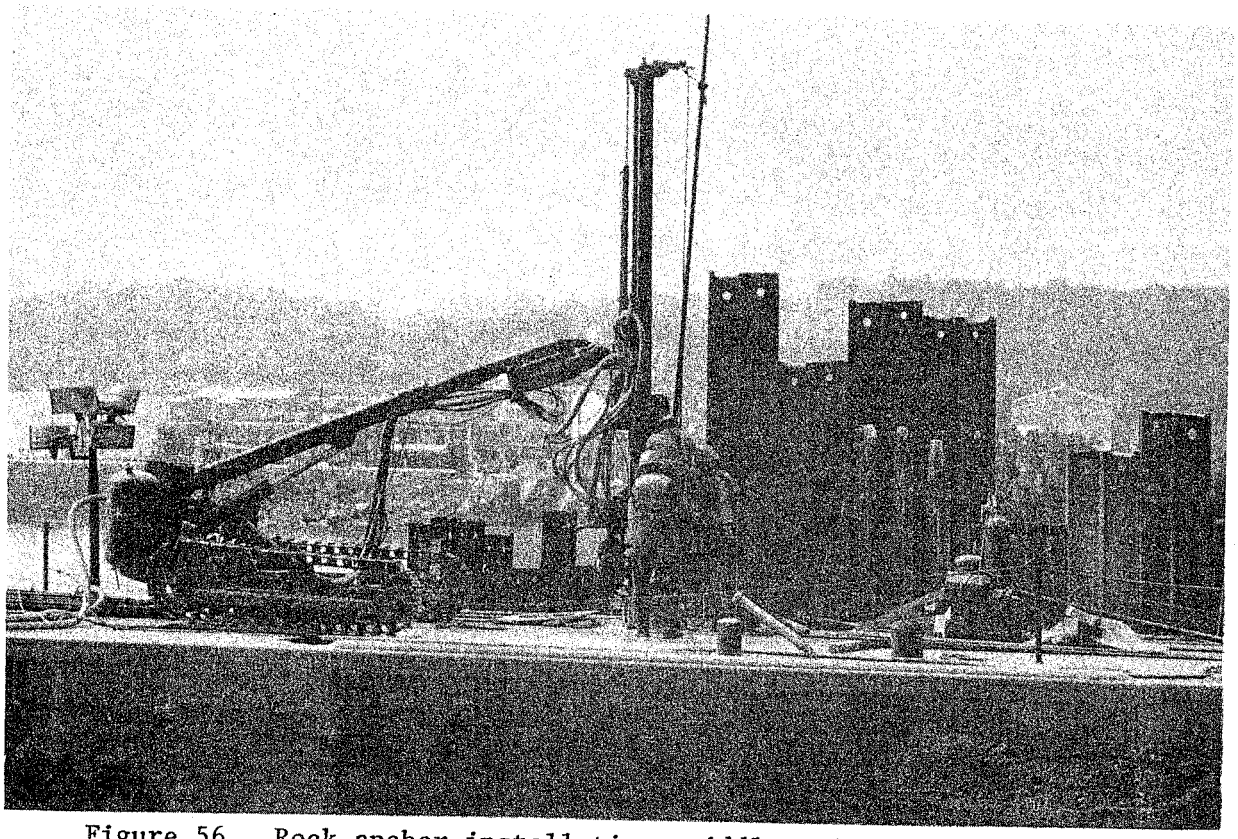


Figure 56. Rock anchor installation, middle wall, September 1978, Lock No. 3, Monongahela River

problems with material in the hole. This arrangement was used for most of the anchors installed.

134. After the drilling was completed, the proper number of polyester resin cartridges was placed in the 2-1/4-in.-diam hole. The anchor bar was then inserted to the top of the cartridges. The contractor did not have a piece of equipment to handle the bars so this work was done manually. The bar was then driven down into the cartridges. The equipment available was capable of spinning and driving the bar for only the last 12 ft (Figure 56). This procedure was not ideal for the installation. Just before the anchors were stressed the secondary grout was tremied into the hole. The anchors were then stressed to the design load and locked off. The lift-off tests were performed as outlined by the Corps.

135. Even after the addition of the 3-in. casing, ECI continued to have drilling problems. It was often difficult to install the casing and remove it prior to anchoring the bar. This problem was probably the result of misaligned holes caused by deflection of the drill steel. The small diameter drill steel was not rigid enough for the long sloped holes. Many holes had to

be abandoned and relocated because of "obstructions" or "voids." The obstructions were old form ties and form lumber which should not have presented any great difficulty for proper equipment. A later inspection of the unwatered river chamber revealed the voids encountered in the river wall were actually a culvert. The location of the culvert on the contract drawings was incorrect. This error combined with the misalignment in drilling resulted in several holes day-lighting in the culvert. Although a few holes were relocated when the culvert was encountered, it was discovered upon inspection of the culvert that many were not. In all, 11 anchors passed through the culvert to varying degrees. Two anchors were cut and new base plates installed because the sections below the culvert floor were grouted. One anchor was cut and no base plate installed when records showed it had been rejected and replaced at another location. Three anchors had casing around them, and no further action was taken. The other anchors, protruding through only the upper corner of the culvert, were coated with a bituminous material and protected with a shroud of concrete. These holes were the cause of large grout takes reported during secondary grouting. From 18 to 24 in. of layered grout (approximately 1,000 cu ft) was found on the culvert floor. It is inconceivable that the drilling crew was not aware of drilling through the culvert; however they proceeded to waste large quantities of grout.

136. Failure of an excessive number of anchors was another very serious problem. The contractor was unable to stress 18 anchors in the river wall and 17 anchors in the middle wall to the design load. The anchors were reanalyzed for possible acceptance with the reduced loading. When the anchor locations were planned, there were many cases in which the exact design spacing was not practical, so additional anchors were included. This increase in anchors later provided some flexibility for reduced loadings. As a result of the reanalysis, 6 of the river wall anchors and 12 of the middle wall anchors were accepted and the rest replaced.

137. A failed anchor was removed from the middle wall and closely examined for any possible explanation of the failure. The general appearance of the bar in the anchorage zone indicated that the polyester resin grout had not bonded to the bar (Figure 57). The lower 5 ft had a light gray material lodged between the deformation of the bar that appeared to be resin grout (Figure 58). However, this resin was soft and pliable and could easily be removed from the bar. In other reaches of the bar, the resin was not soft,

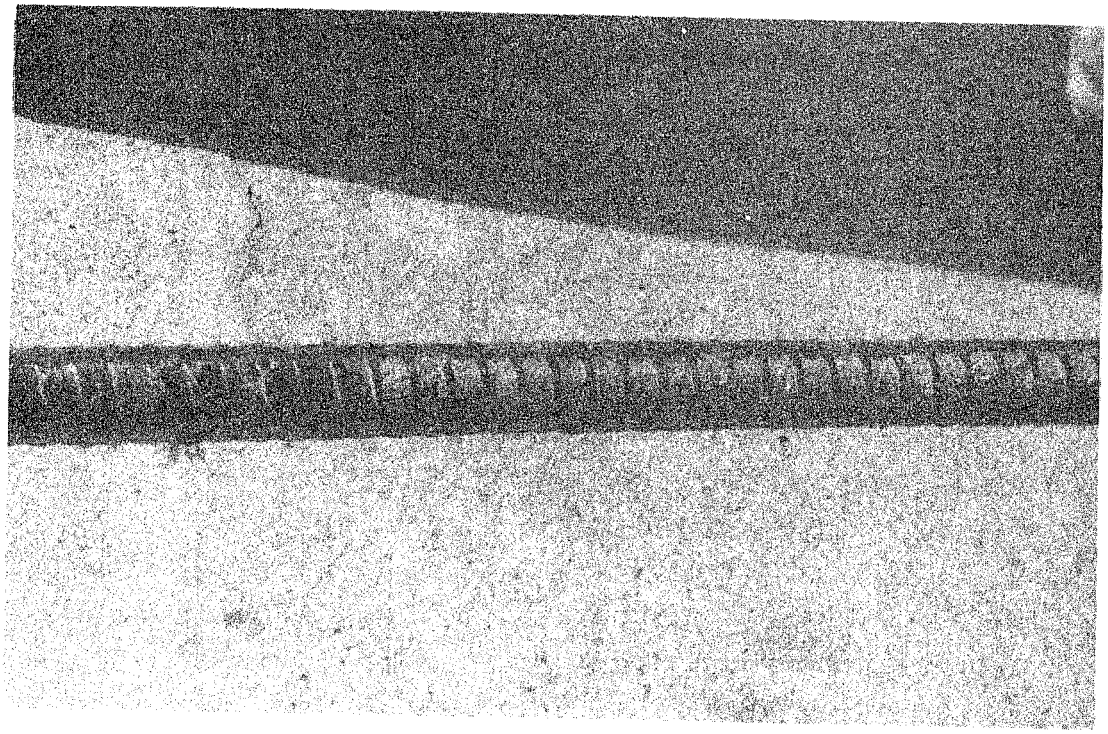


Figure 57. Typical condition of anchorage zone of failed anchor, Lock No. 3, Monongahela River

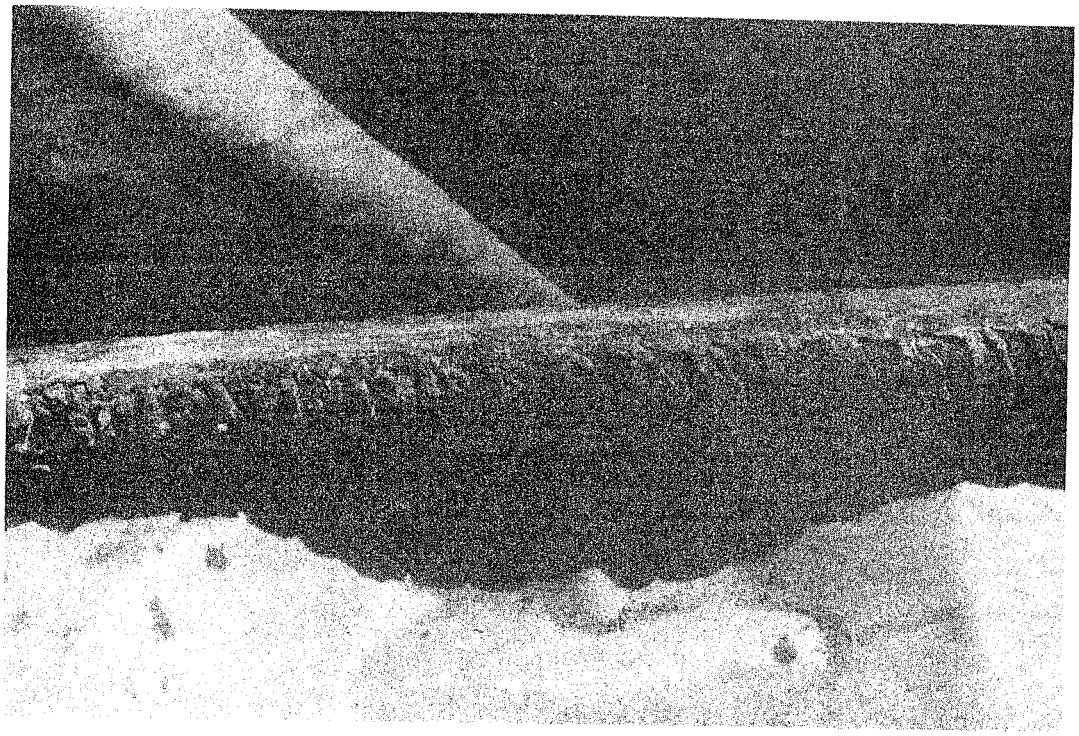


Figure 58. Soft grout lodged between bar deformations, Lock No. 3, Monongahela River

and it was harder to remove from the bar. The contractor claimed that improper mixing occurred because the hole was enlarged by the caving of the hole in the poor rock which would not have happened if he had been allowed to pre-grout each hole. To determine if the 2-1/4-in.-diam hole was possibly being enlarged during drilling, the hole from which the failed anchor was removed was grouted with a reddish grout and a core boring was taken (Figure 59). The core showed that the hole was consistently 2-1/4 in. in diameter.

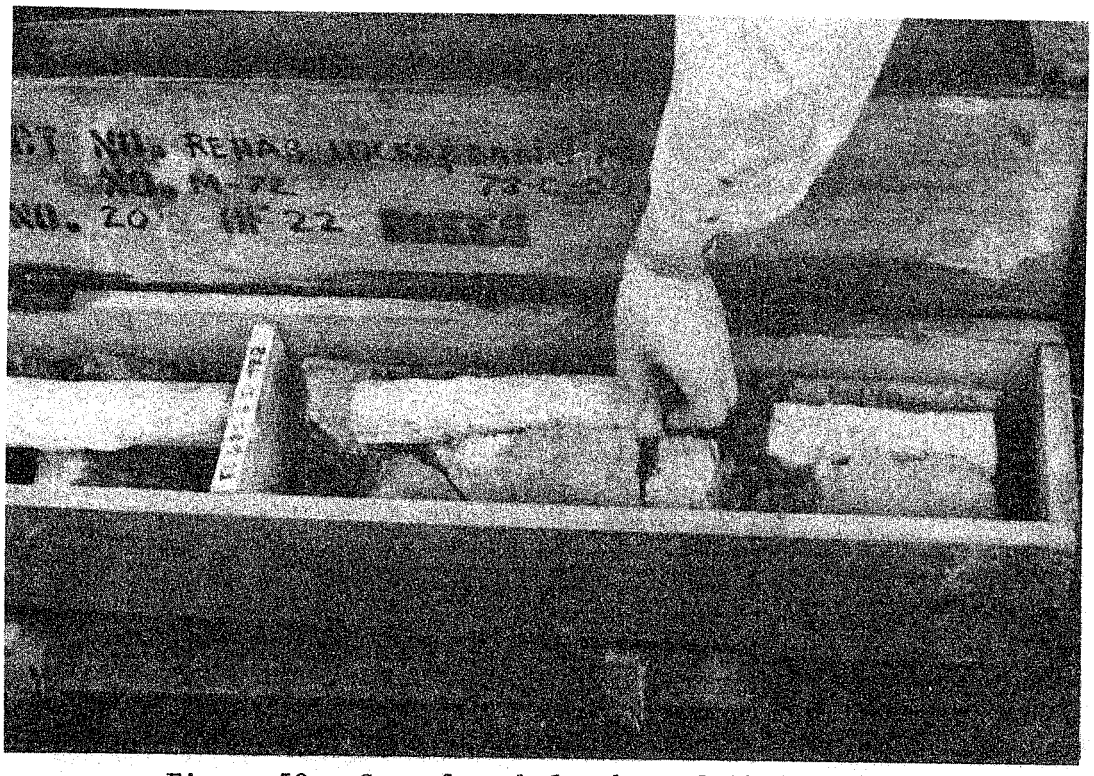


Figure 59. Core from hole where failed anchor was removed, Lock No. 3, Monongahela River

138. In the interest of better consistency and progress in the anchor installation, the Corps recommended a cement grout system be used to anchor the bars. In March 1979, the contractor changed his procedures to drilling a 4-1/2-in.-diam hole full length and using cement grout. The anchors were tensioned after 9 days, and the stressing length was grouted. This method produced more consistent results and far fewer failures and thus was used to install approximately one-fourth of the anchors on the middle wall and three-fourths of the anchors on the river wall (Figure 60).

139. In addition to the anchors originally included in the contract, nine anchors were added by change order in November 1978. After the cofferdam for the new lower miter sill was pumped out, an inspection of the foundation

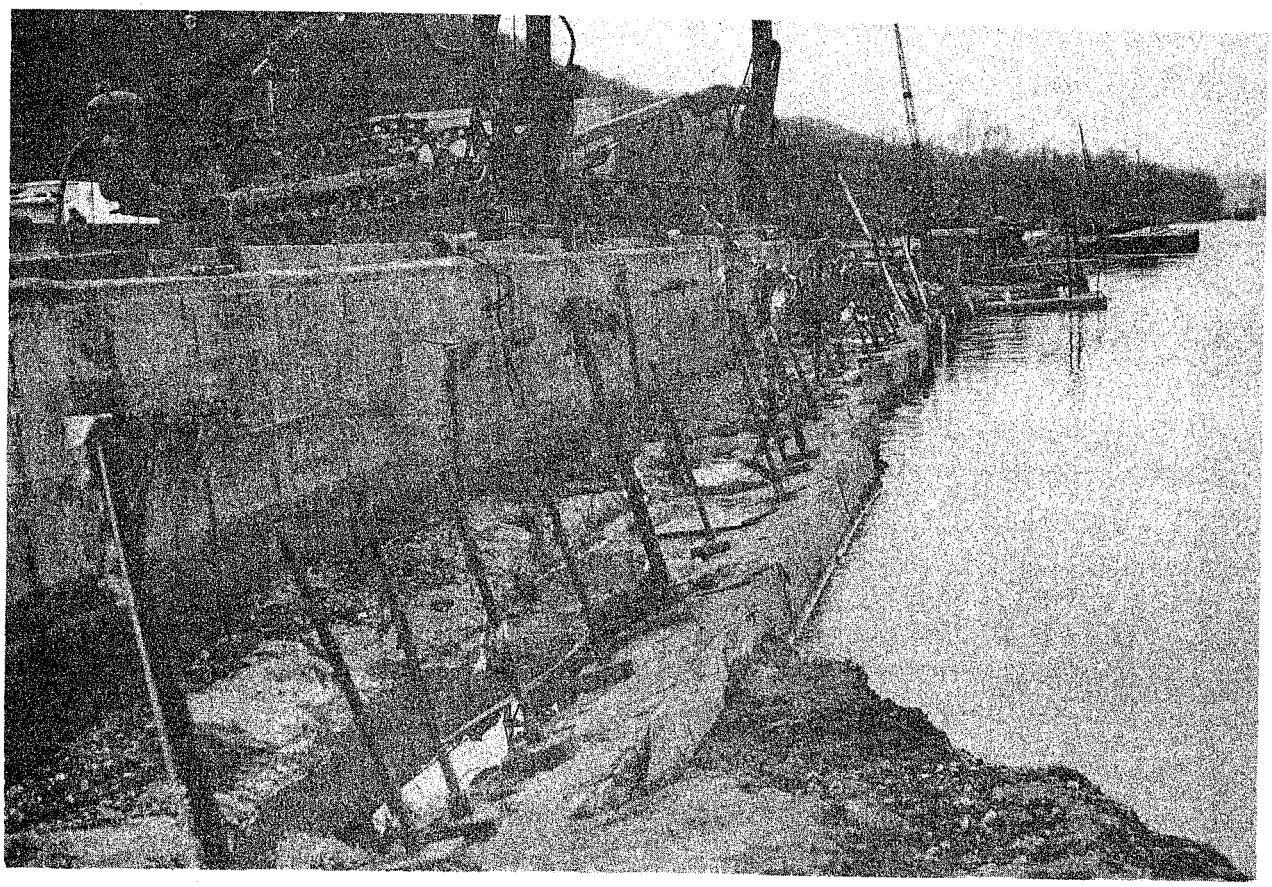


Figure 60. Rock anchor installation, cut down top of river wall, March 1979, Lock No. 3, Monongahela River

rock revealed the parameters used in the stability analysis may not have been justified. To ensure adequate stability, nine anchors were installed in monoliths M-25, M-26, and M-27. The anchors were installed through the river face of the wall on a 45 deg angle, 25 ft into rock. Celtite polyester resin grout cartridges were used to anchor the bars, and cement grout was used for secondary grouting. The bars were stressed to 15 kips.

140. In April 1979, the Pittsburgh District requested approval from the Ohio River Division to delete from the contract the rock anchors in the upper and lower guide walls. Because of the contract administration problems being experienced, the need for anchors was reevaluated. Although the walls do not meet even the reduced requirements established for Lock No. 3, their performance has been highly satisfactory as evidenced by the absence of movement. Should any movement occur in the future, corrective action could be taken. The request was approved, and a change order was issued in July 1979.

141. Originally, the 23 anchors in the land wall were designed to be installed through the chamber face of the wall and the work to be accomplished

during Phase II when the land chamber was closed. The contractor proposed to install the anchors through the top of the wall to allow the work to be done during Phase I. Since this method of installation would change the angle of the anchor, a greater load per bar was required. The contractor chose to use 1-1/4-in.-diam bars at his own expense instead of the 1-in.-diam bars specified. This change enabled him to increase the stress per bar, so the proposal was approved. All 23 land wall anchors were installed using the cement grout method.

142. Although the desired stabilization of the lock walls was achieved, the overall performance of the work must be considered less than satisfactory. In many areas, the contractor's methods and equipment were inefficient, and the drilling crews could not cope with the problems encountered. The small diameter drill rods used were too flexible for the lengths required. A larger rotary drill rig capable of handling 3-in.-O.D. drill rods and a down-the-hole hammer would have eliminated many of the problems. There was no equipment onsite for handling the long anchor bars which were placed manually with the aid of a pulley on the drill rig. As stated earlier, the drilling crews drilled through voids without reporting them and then proceeded to waste large quantities of grout. The contractor continually complained about "caving" in the holes. The core borings taken in the failed anchor holes showed the walls to be stable. The deposits causing the problems were actually the drill cuttings which the contractor was unable to expel from the hole.

143. The original design and procedures established by the Pittsburgh District provided a sound scheme for anchoring the walls using a cement grout method. When the contractor chose the resin grout option, he failed to modify his procedures accordingly. The polyester resin cartridge manufacturer, Celtite, recommended a 2-1/4-in.-diam hole for proper mixing of the 45-mm cartridge with a 1-1/4-in.-diam bar. Subsequent testing by WES determined that the hole size was borderline on being too large and the semihardened condition of the hardening agent required a greater amount of mixing than specified by the resin manufacturer's literature. This condition may have contributed to the large number of failures encountered with the polyester resin grout system. From the results obtained and the problems experienced, it was concluded that the cement grout method for bonding the anchors is better suited for this type of installation.

144. To accommodate the normally high volume of commercial traffic

while the large land chamber was closed during the Phase II work, the 360-ft-long river chamber was extended to 720 ft. This extension was accomplished by constructing a sheet-pile cell river wall extension and a new lower gate monolith and miter sill during Phase I (Figure 48). The sheet-pile cell wall consists of eight circular cells, 30.8 ft in diameter, connected by seven arcs. The length of this wall is 325.7 ft. Used Government-furnished sheetpiling, PS-28 driven to rock, was used for the cells. The cell fill is clean sand and gravel dredged from the Monongahela River at the project site which met the required gradation. Each cell is capped with 1 ft of concrete reinforced with two layers of 6 by 6 - 4.0 w by 4.0 w wire mesh. The concrete, the contractor's 5-bag mixture, was mixed onsite in the contractor's floating batch plant. Three rows of 3/4- by 10-in. armor plate were placed on the chamber side of each cell to act as rubbing strips and protect the interlocks.

145. A U-shaped steel sheet-pile cell cofferdam attached to the middle wall was used to construct the new river wall monoliths, R-24 and R-25, and the new miter sill, in the dry (Figure 61). Bedrock was excavated for the foundation. Wall monoliths R-24 and R-25, which are 61.5 ft high, were placed

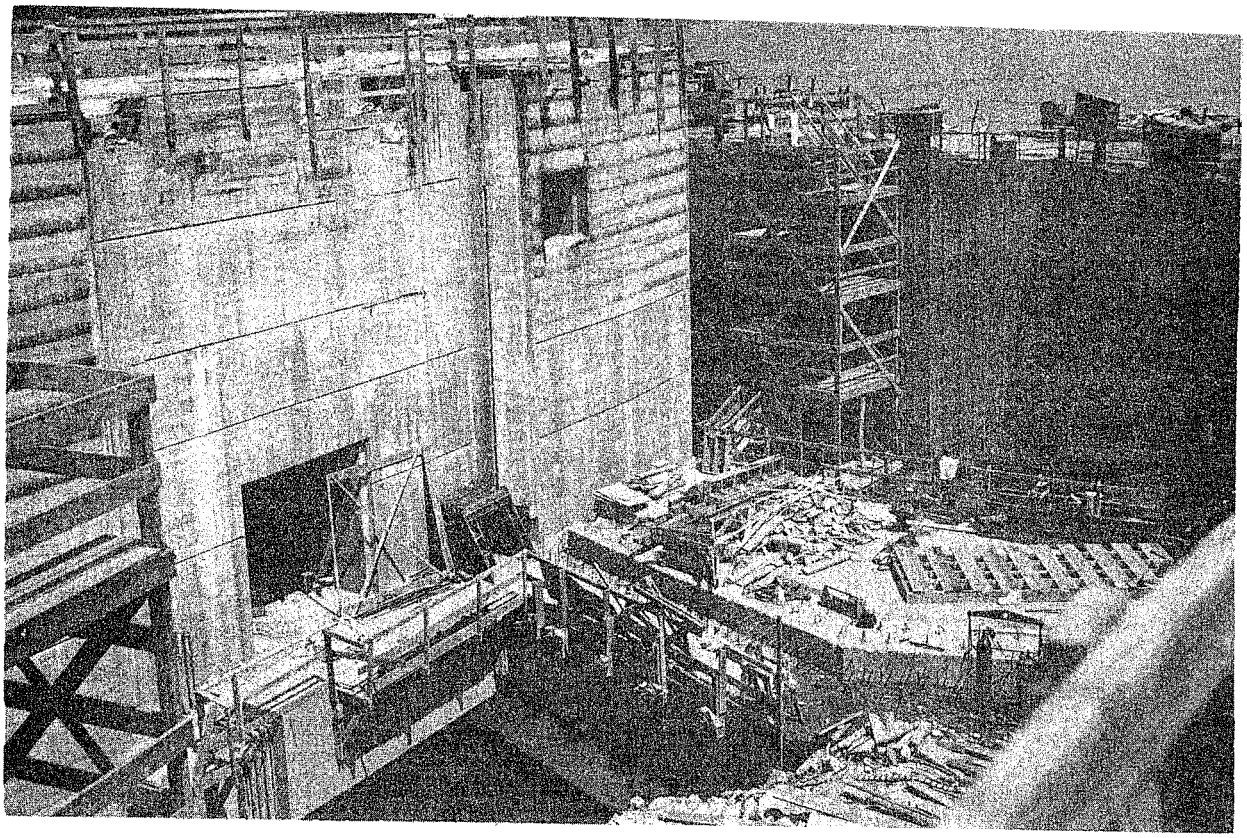


Figure 61. Construction of new river wall monoliths and miter sill, August 1979, Lock No. 3, Monongahela River

in 10 lifts. The concrete was green cut between all lifts. The concrete for the monoliths and the miter sill was the contractor's 5-bag mixture. To increase the emptying capacity of the extended chamber an 8- by 8-ft vertical slide gate was installed in monolith R-24 (Figure 61). The slide gate is operated by a vertically mounted hydraulic cylinder.

146. To provide a miter gate recess in the middle wall, the river face of monoliths M-24, M-25, M-26, and M-27 was modified (Figure 62). One foot of

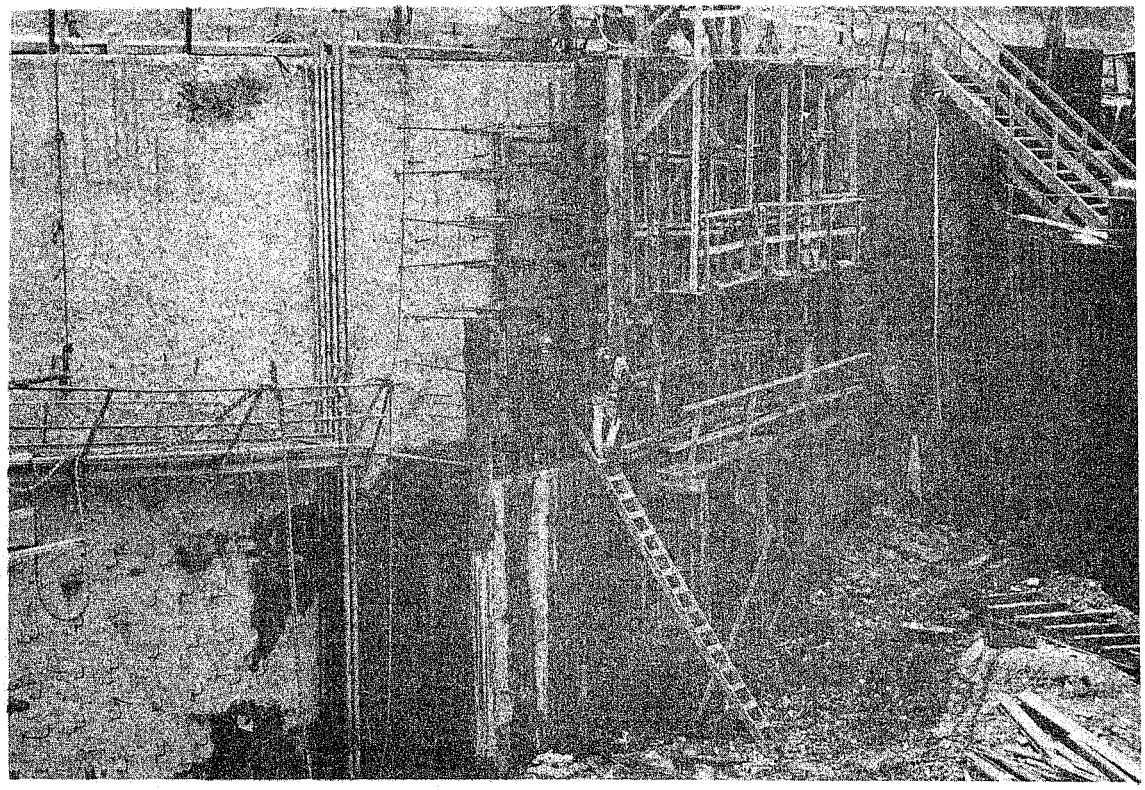


Figure 62. Construction of new downstream gage recess in river face of middle wall, August 1979, Lock No. 3, Monongahela River

concrete was first removed from the face by line drilling and blasting some sections and chipping others with a Hoe Ram impacter mounted on a backhoe. The monoliths were then refaced with 1 ft of concrete at the ends to meet the existing walls and 4 ft in the area of the new gate recess. The replacement concrete was anchored to the existing wall with No. 6 and No. 8 dowels on 2-ft centers and reinforced with a vertical mat of No. 5 bars on 1-ft centers. After 4.5 ft of concrete was removed from the top of the wall, the new gate machinery and anchors were installed and new concrete was placed.

147. Two sets of approach cells were constructed downstream of and in line with the middle wall. Each set consisted of two circular sheet-pile

cells, 19.1 ft in diameter, connected by an arc cell. The sheetpiling was Government-furnished PS-28 piling driven to top of rock. The cells were filled with tremie concrete to 1 ft above lower pool, and the remainder was filled with regular concrete. The concrete was mixed onsite in the contractor's floating batch plant. The tremie concrete was the contractor's 7-bag mixture, and the regular concrete was the 5-bag mixture. One layer of 6 by 6 - 4.0 w by 4.0 w wire mesh reinforcing was placed approximately 2 in. below the surface of the concrete.

148. The river chamber extension was placed into operation in May 1980 (Figure 63). Initially, there was some concern about the sheet-pile cells

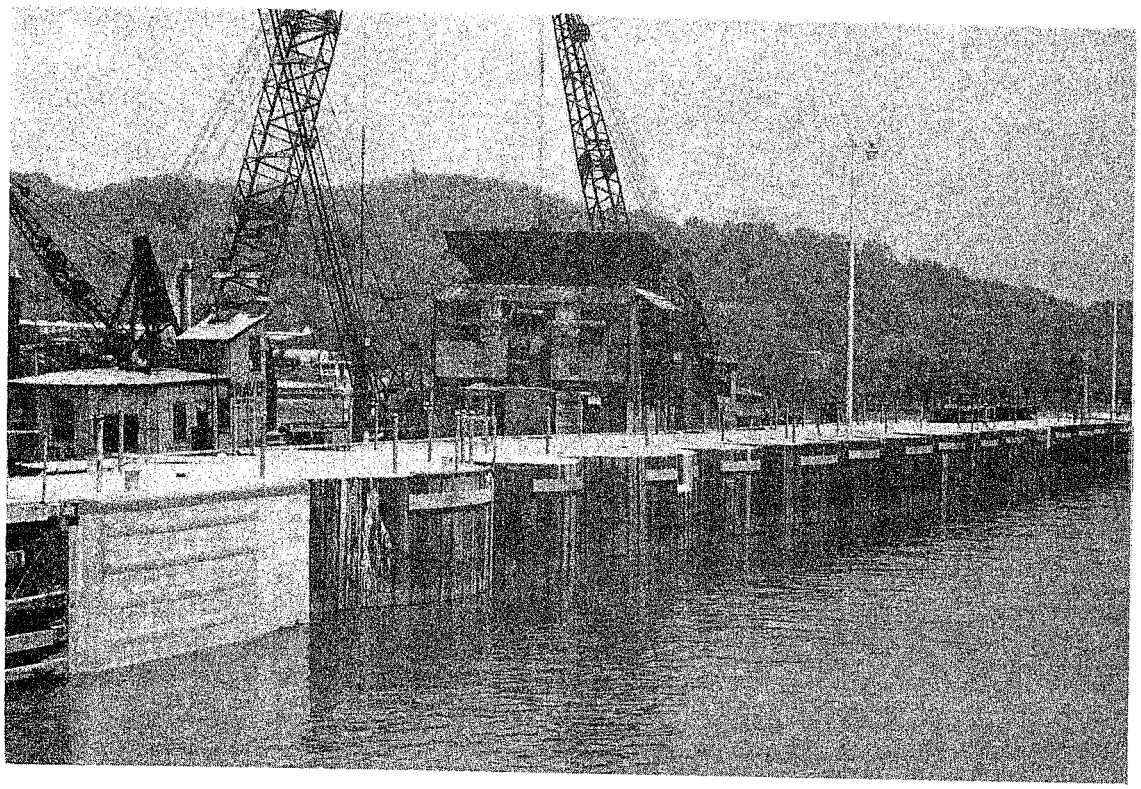


Figure 63. Completed river wall extension, May 1980, Lock No. 3, Monongahela River

when the concrete caps on most of the cells and arcs settled to as much as 6 in. on the riverside. A diving inspection was conducted to determine whether cell fill was being lost because of parted interlocks or erosion below piling. The divers found neither condition, and it was concluded that the settlement was caused by fill compaction and would cease.

149. Since the completion of the rehabilitation, the District has requested and obtained permission to leave the extension in place. The river chamber will be operated as a 360-ft-long chamber since the land chamber

emptying valves discharge into the area of the extension making the simultaneous use of two large chambers impossible. As the extended chamber does not appear to hinder navigation, the District feels that the retention of the extension will prove beneficial for use as a large chamber should an emergency arise. Since this request has been approved, an added benefit will be saving the estimated cost of $936,000 for removing the temporary river chamber extension and an estimated cost of $120,000 for removing the lower approach cells.

150. The lock walls were badly spalled and scoured, and the concrete surfaces were weathered and deteriorated. The worst conditions were between pool levels and at monolith joints (Figure 64). The concrete under the gate

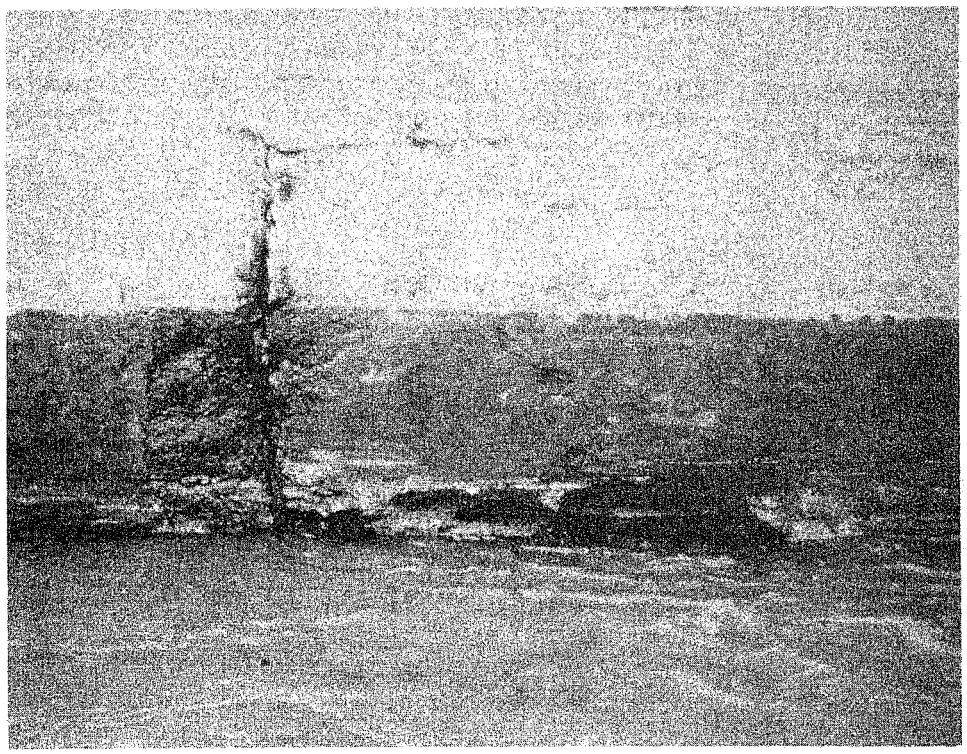


Figure 64. Lock wall concrete deterioration, 1974, Lock No. 3, Monongahela River

machinery was cracked and deteriorating. The guide walls were abraded and scoured by tows entering and leaving the locks. Previous wall repairs were deteriorating or spalled off (Figure 47). Core borings had shown poor concrete existed for 2 to 6 ft from the surfaces. To prevent further deterioration of the walls, it was necessary to protect the concrete which was still sound. In some areas, this protection was accomplished by removing a specified thickness of old concrete and replacing it with new reinforced concrete.

In other areas, a new surface was placed over the old, using shotcrete for vertical faces and reinforced concrete for tops of walls. See Figures 55 and 65 for the designed treatment for each wall.

151. The majority of the concrete removal work was for the vertical faces of the lock walls at various locations. To remove the concrete, a scheme of line drilling (Figure 66) and blasting was developed. In 1978, two test blasts were conducted on the river wall to determine the most efficient blasting parameters. Blast holes were 2 in. in diameter, 19 ft deep, and 1 ft from the face of the wall. Each hole was loaded with two or more lengths of 50-grain/ft detonating cord taped along opposite sides of a 1-1/2-in.-diam wood pole. Stemming in the collar was wet sand. The first blast involved four holes with 200 grains/ft and three holes with 150 grains/ft. The holes were on 12-in. centers. The second blast involved 13 holes on 6-in. centers using 100 grains/ft. Four transducers were located in the vicinity of the blasting area to record vibration intensity. Transducer No. 1 was located on the test monolith, directly behind the blast and 1 ft from the outside face of the wall; No. 2 was on the adjacent downstream monolith; No. 3 was on the adjacent upstream monolith; and No. 4 was on the middle wall monolith directly across from the test monolith. Test data are summarized as follows:

Blast No.	No. of Holes	Explosive (grains/ft)	Spacing in.	Transducer Location	Particle Velocity (ft/sec)
1	4	200	12	1	4.3
	3	150	12	2	7.2
				3	4.0
				4	0.4
2	13	100	6	1	6.6
				2	2.8
				3	4.9
				4	0.2

The three test procedures each produced clean breaks with no damage to the remaining structure. From these results, a decision was made to try 100 grains/ft at 12-in. spacing for production blasting.

152. Blasting for removal of concrete within the chambers was done with water at lower pool level to help dampen the effects of the blasting. The selected charge of 100 grains/ft worked well above the waterline but produced inconsistent results below the waterline. To compensate for the cushioning

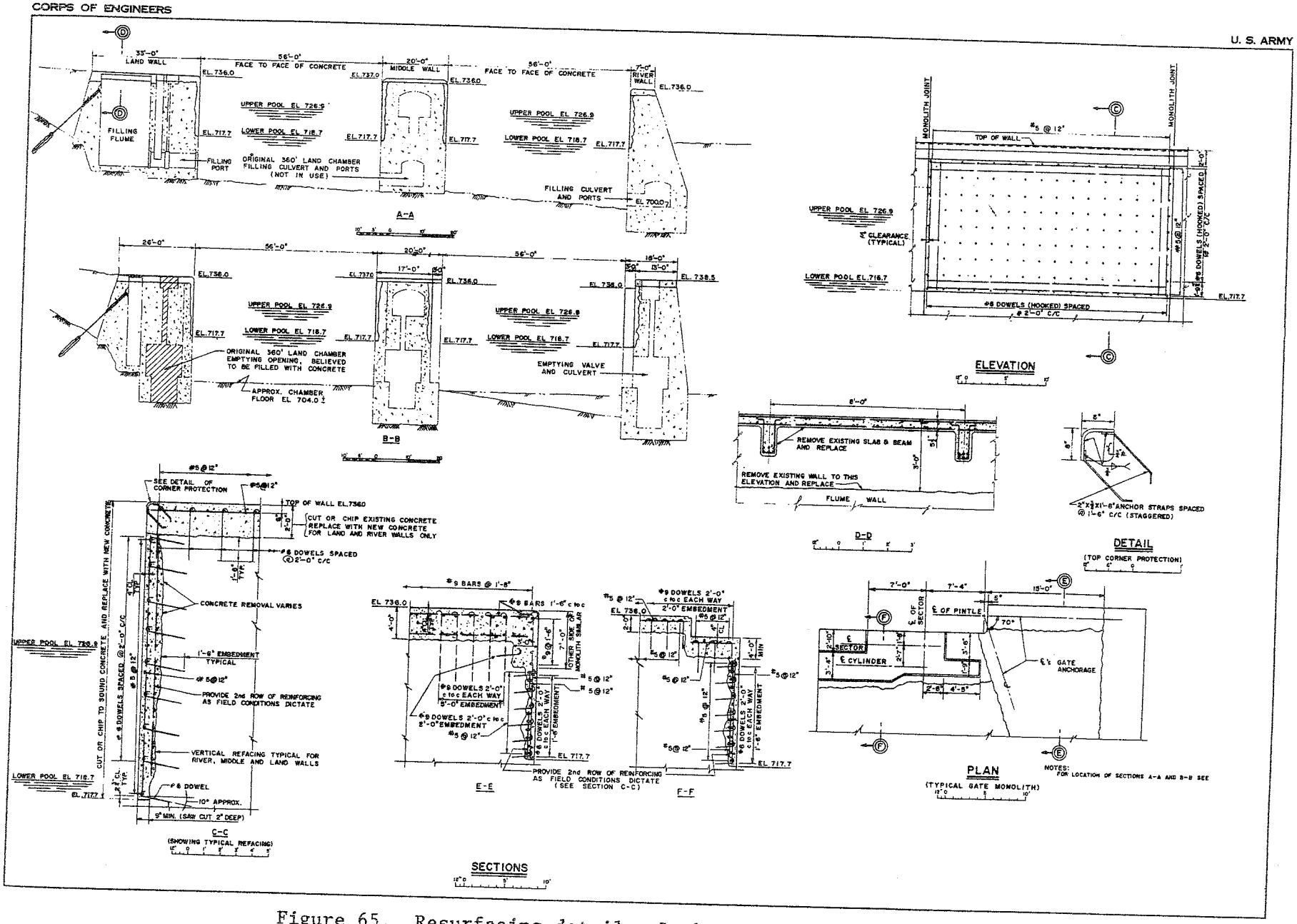


Figure 65. Resurfacing details, Lock No. 3, Monongahela River

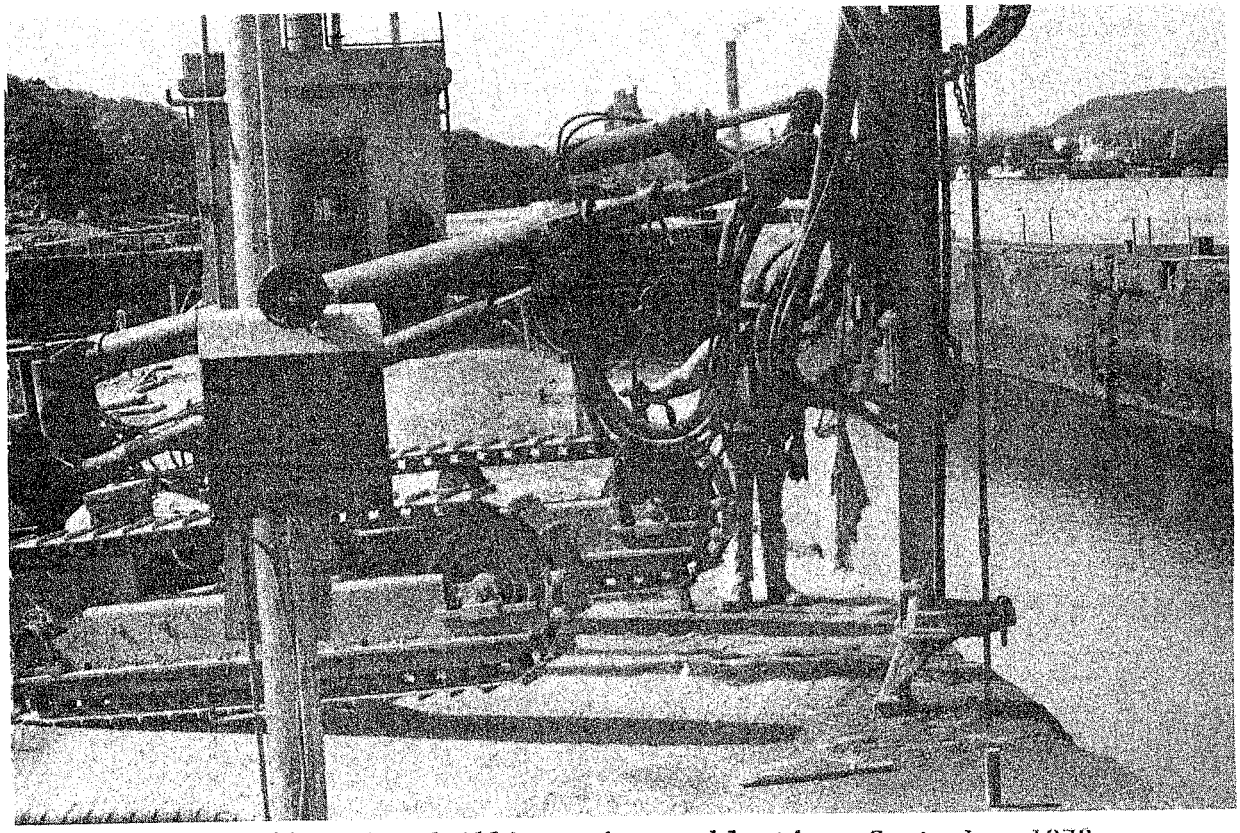


Figure 66. Line drilling prior to blasting, September 1978, Lock No. 3, Monongahela River

effect of the water, a charge of 200 grains/ft was used below the waterline and the desired results were achieved. Concern over the integrity of the monoliths prompted the Corps to request that the contractor try millisecond delays between holes. This procedure was found to leave high spots between the holes and thus was abandoned.

153. The only major problem with the concrete removal was in meeting the dual requirements of 12 in. of new concrete and rebuilding the walls to existing lines and grades. The vertical blast holes were drilled 12 in. from the top edge of the lock wall. It was later discovered that the walls were not plumb and less than 12 in. was removed at the bottom. This discovery required unplanned chipping to achieve the desired cover. The contractor used jackhammers and a Hoe Ram mounted on a backhoe for this work. In the future, blast holes should be drilled farther from the face of the wall and the new wall faces constructed plumb to avoid additional concrete removal work. Overall, the use of blasting to remove concrete from the wall faces proved to be a very efficient method. This method allows a contractor to do the major part of the removal work, line drilling, without closing the lock chamber. Where

minimizing lock downtime is critical, this procedure can be significant.

154. Originally, the top surfaces of the land and river walls were to be cut down 2 ft and the top of the middle wall cut down 1 ft. When work began on the middle wall, it was discovered that there was inadequate cover over the gallery for removal of 1 ft of concrete. Instead, the middle wall was capped with 1 ft of concrete. The contractor later proposed as a VE proposal the same treatment for the land wall and the lower portion of the middle wall, and it was approved. The only areas to be cut down on the land and middle walls were under the gate anchorages and operating machinery. The river wall was the only wall to be cut down and resurfaced to the original elevation. Removal of the top surface concrete was done with jackhammers and a Hoe Ram mounted on a backhoe (Figure 67).

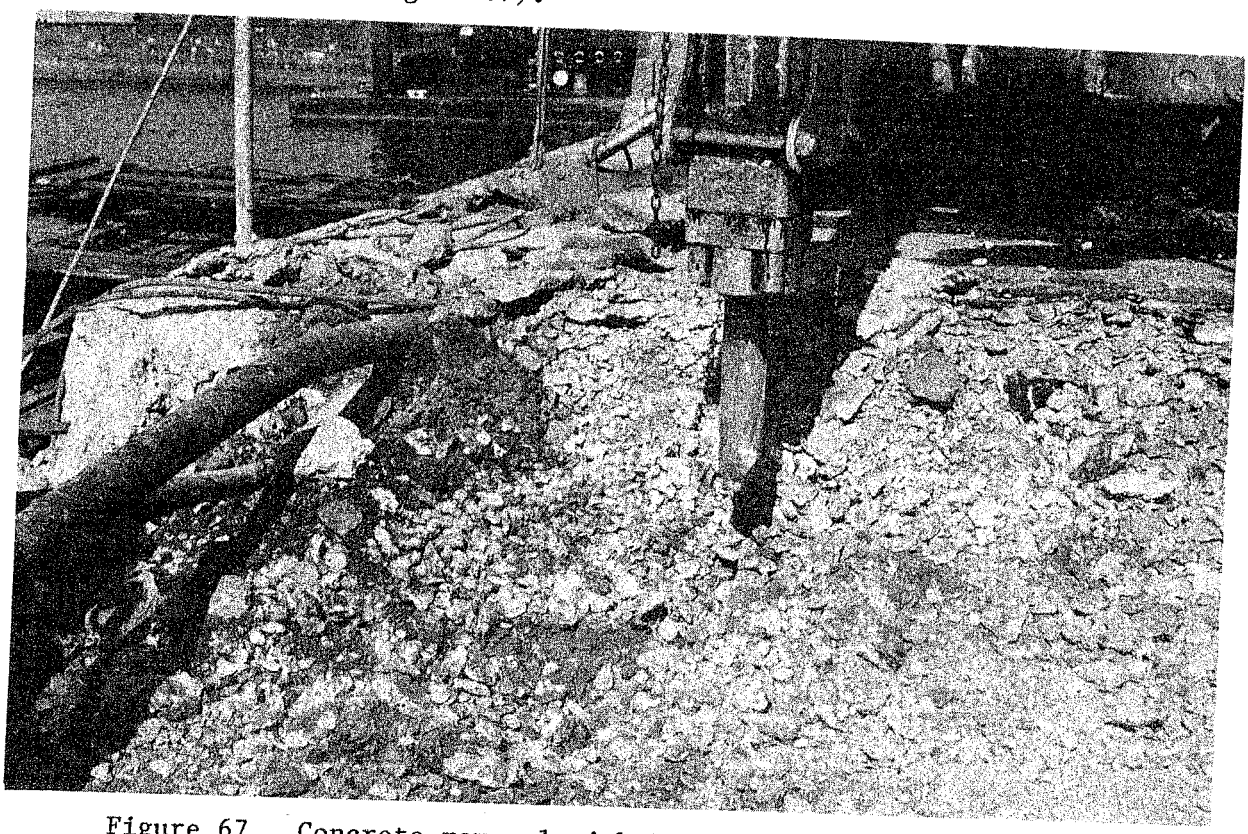


Figure 67. Concrete removal with Hoe Ram Impactor, October 1978, Lock No. 3, Monongahela River

155. After the existing concrete was removed from the face of the walls (Figure 68), 1-1/2-in. holes inclined 10 deg below horizontal were drilled on 2-ft centers for No. 6 hook dowels (Figure 69). The dowels were grouted in place with polyester resin grout (Celtite) cartridges. A cartridge was inserted into each hole, and the dowel was installed and spun with an air drill.

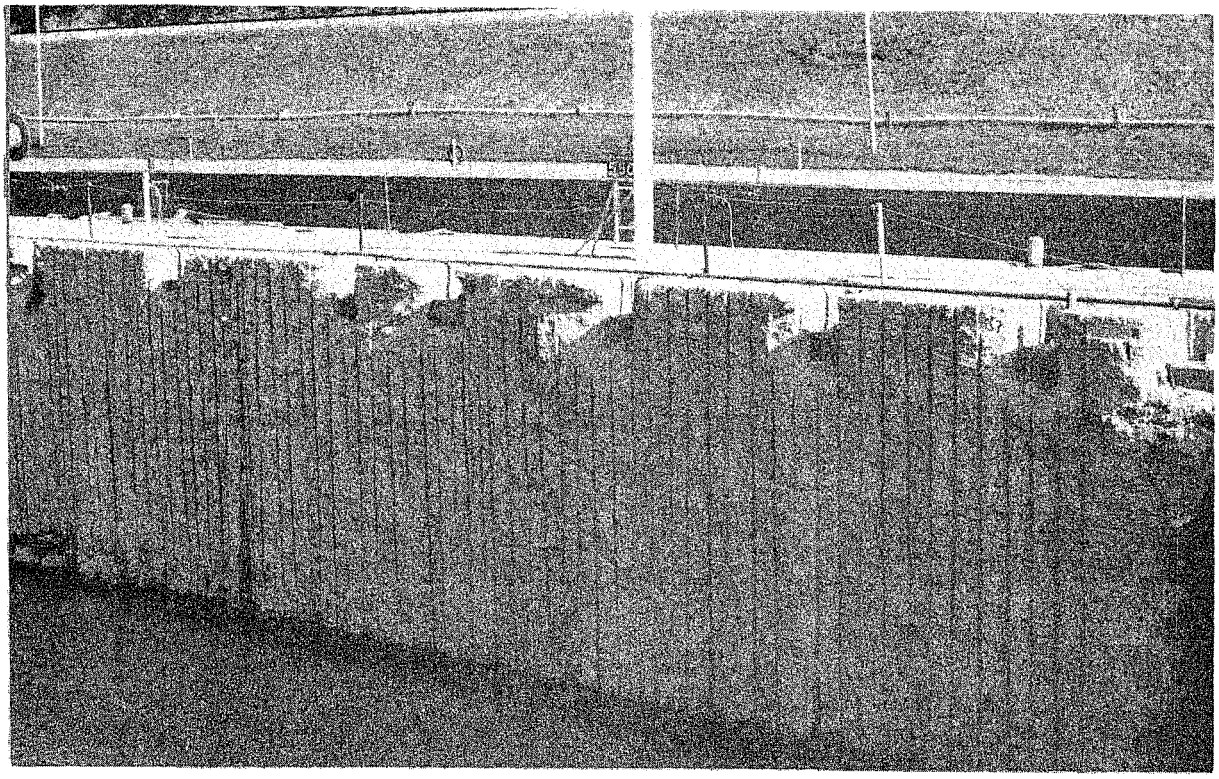


Figure 68. River face of middle wall after blasting, October 1978, Lock No. 3, Monongahela River

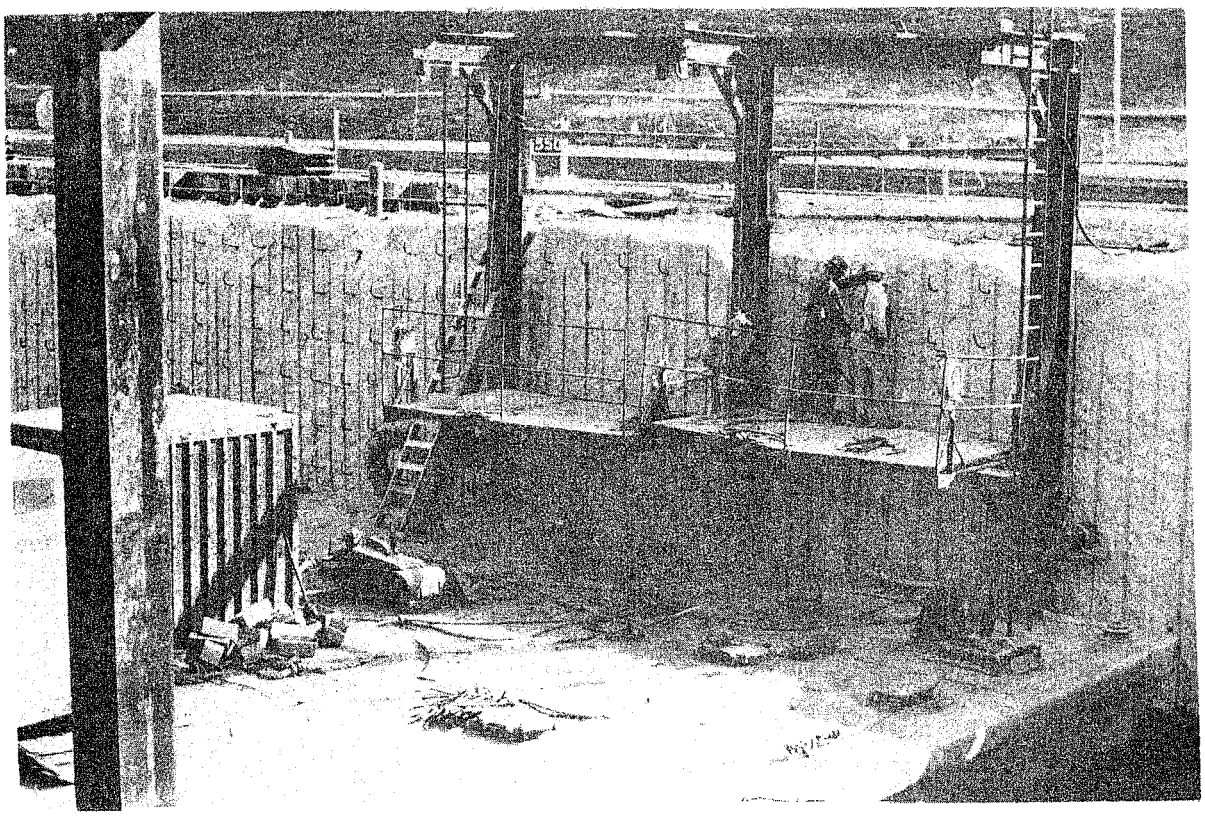


Figure 69. Installation of dowels, November 1978, Lock No. 3, Monongahela River

A vertical mat of No. 5 reinforcing bars on 12-in. centers was then positioned approximately 4 in. from the eventual face of the wall (Figure 70). Forms were installed, and the concrete was placed using an 8-in.-diam, nonrigid, and PVC tremie pipe. A collapsible pipe was needed because of the narrow spaces around the reinforcing steel. The same dowel installation procedures and reinforcing mats were used when the tops of the walls were resurfaced.

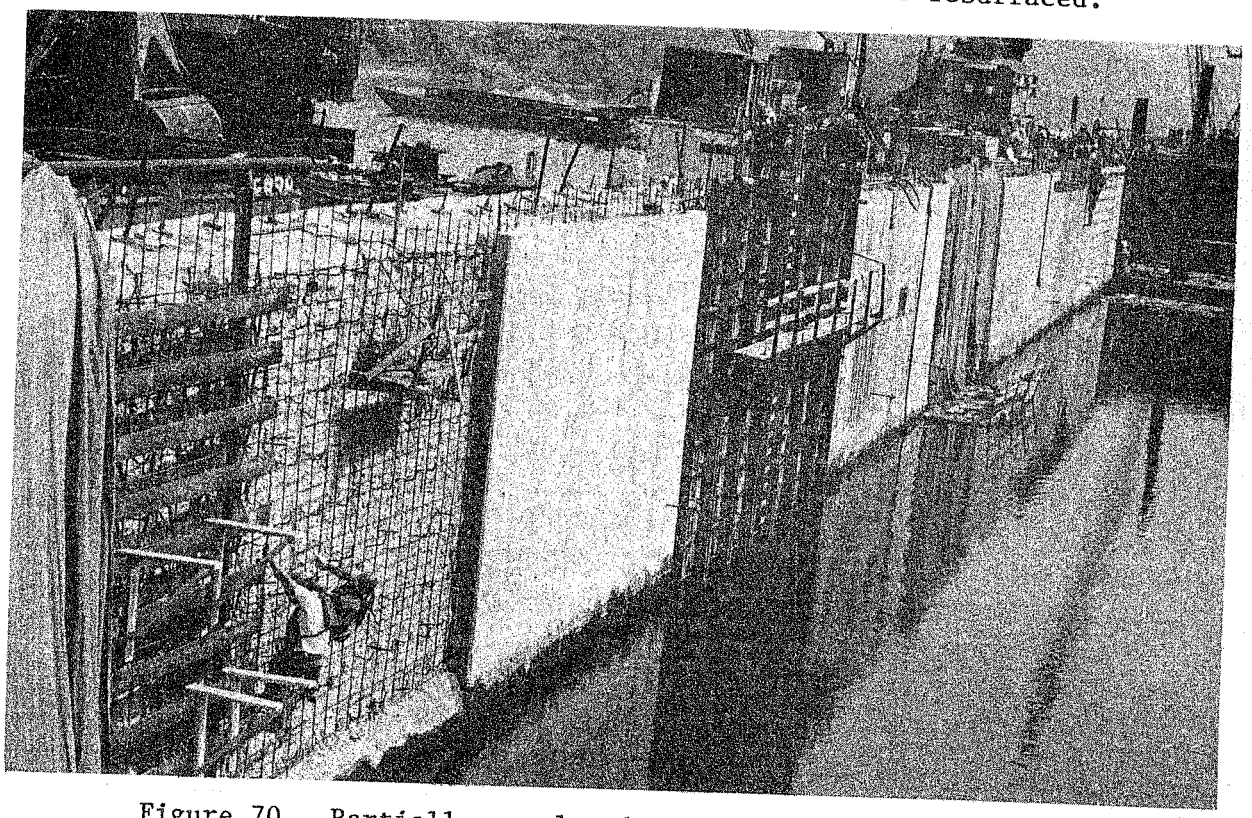


Figure 70. Partially completed land face of river wall, August 1979, Lock No. 3, Monongahela River

156. Two concrete mixtures were used during the rehabilitation work. Mixture 1A, which was used for all wall repairs above water, was a 5-bag mixture with a water-cement ratio of 0.49 by mass and 7.0 percent air entrainment. The coarse aggregate met the Pennsylvania Department of Transportation 2B grading with 1-1/2-in. maximum size. Mixture 2, which was a tremie concrete mixture used for all underwater placements, was a 7-bag mixture with a water-cement ratio of 0.42 by mass and 5.0 percent air entrainment. The coarse aggregate was the same as in Mixture 1A. Natural sand, crushed limestone coarse aggregate, and Type II portland cement were used. Batch water was obtained from the system of the West Penn Water Company.

157. The concrete was mixed in the contractor's floating batch plant. The batch plant had two 4-cu yd mixers. There were bins for two sizes of

coarse aggregate, sand, and cement. The plant contained automatic scales for batching. A sensor in the sand bin monitored the moisture content, and the batch water was automatically adjusted. The air-entrainment admixture was applied from a dispenser controlled by a timer. The dispenser was initially calibrated for the correct amount of admixture and then periodically checked. A local testing laboratory checked the performance of the mixers, and the results were satisfactory. Normally, only 2 cu yd of concrete was batched with a mixing time of 1-1/2 min. The use of a floating batch plant, which could be moved to the area of the placement, minimized the time between mixing and placing and thus eliminated the need to handle the concrete more than once. A derrick boat was used to move the concrete buckets directly from the batch plant to the placement.

158. Fresh concrete surfaces were sprayed with Type I Concrete Curing Compound, Resin 30, supplied by George Wilson Company of Pittsburgh, Pennsylvania. Expansion joint material was 1/2-in. sponge rubber, manufactured by Sealtight. The joints were sealed by first applying a primer, N-49, and then the sealer, Chem-Calk 550, both manufactured by Woodmont Products, Incorporated, Huntingdon Valley, Pennsylvania. Typical examples of the resurfaced concrete are shown in Figures 71-73.

159. The riverside of the river wall, the lower guide wall, all existing gate recesses, and various areas on the upper guide wall and middle wall were treated with shotcrete. The wall faces were prepared by removing the deteriorated and loose concrete with air chipping tools and a bush hammer head mounted on a Hoe Ram (Figure 74). Some wall surfaces required only sandblasting. A high-pressure water jet was then used to clean and wet the surface. Shotcreting was done using the dry-mix method with water added at the nozzle. The sand/cement ratio was approximately 4:1.

160. A thin overlay of mortar (SikaTop 122) was placed on the filling flume deck slab upstream of the powerhouse. The mortar is described by the manufacturer, Sika Corporation, as a polymer-improved, cementitious, two-component, test-settling trowel grade, easy-tool patching mortar excellent for horizontal and vertical surfaces.

Renovation of operating features

161. A new hydraulic oil system including pumps, piping, valves, and cylinders was installed. Three variable volume, pressure-compensated, 30-gpm vane type pumps (Racine model No. PSV-PNSO-40HRM) were installed. The pumps

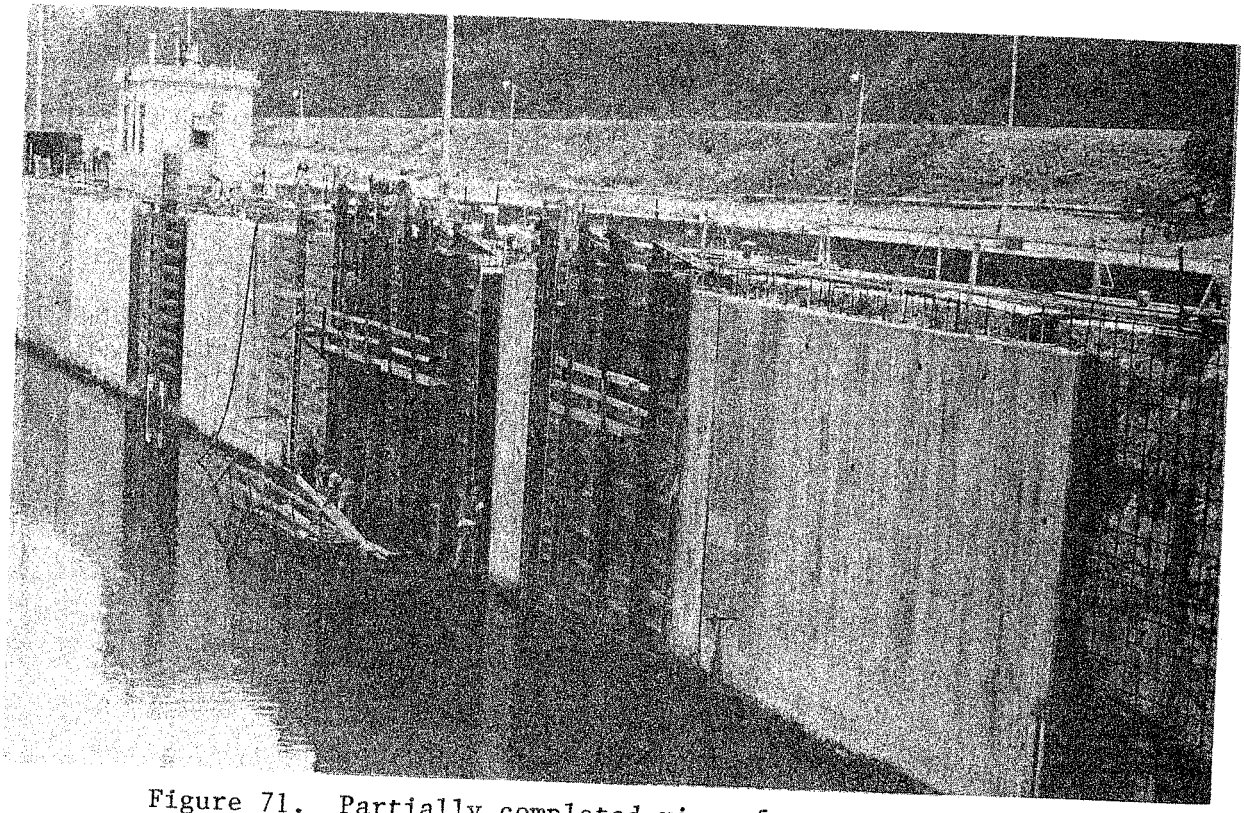


Figure 71. Partially completed river face of middle wall, August 1979, Lock No. 3, Monongahela River

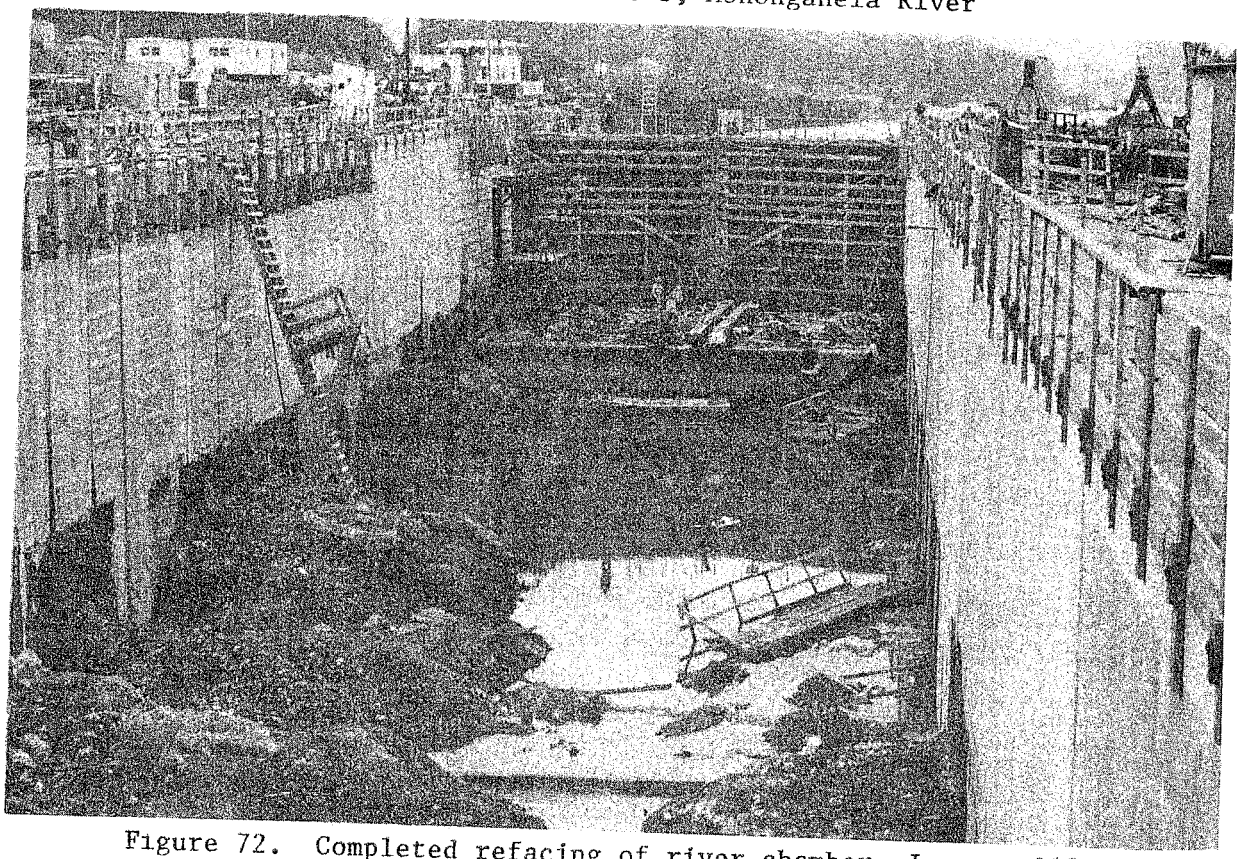


Figure 72. Completed refacing of river chamber, January 1980, Lock No. 3, Monongahela River

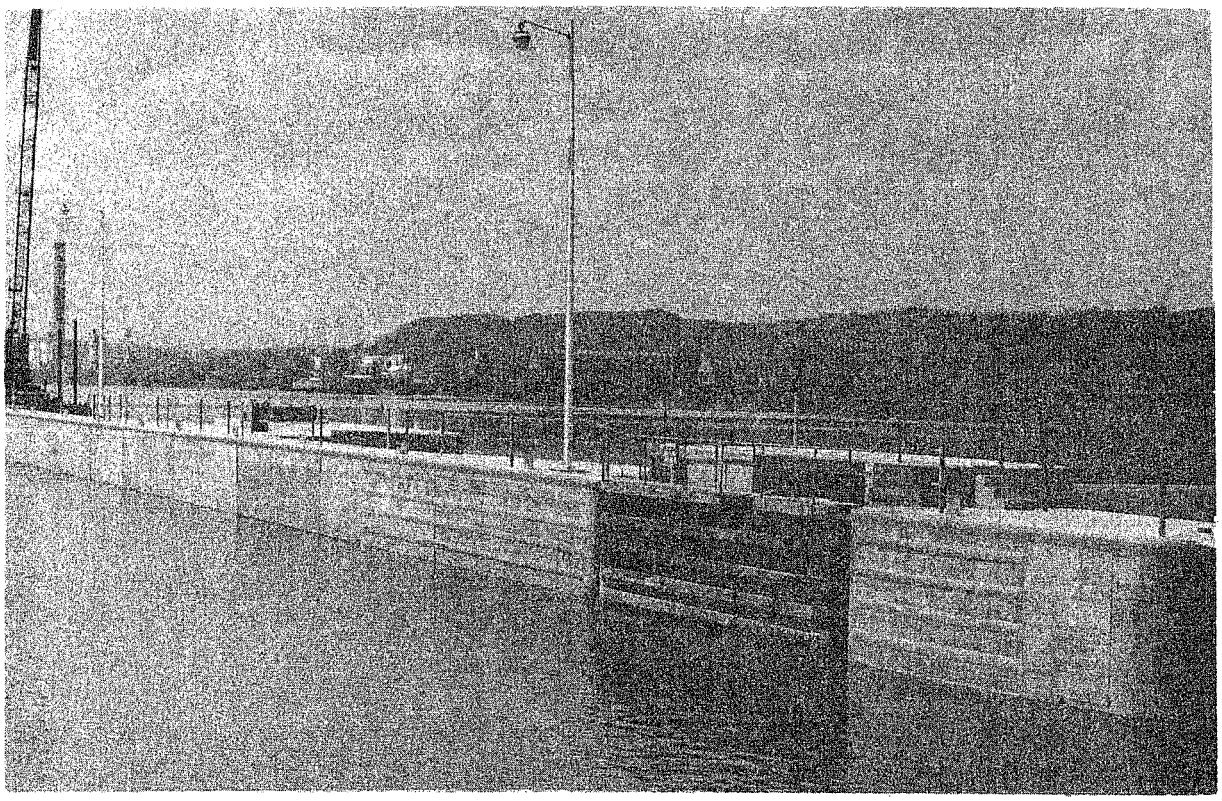


Figure 73. Resurfaced and refaced river wall, May 1980, Lock No. 3, Monongahela River

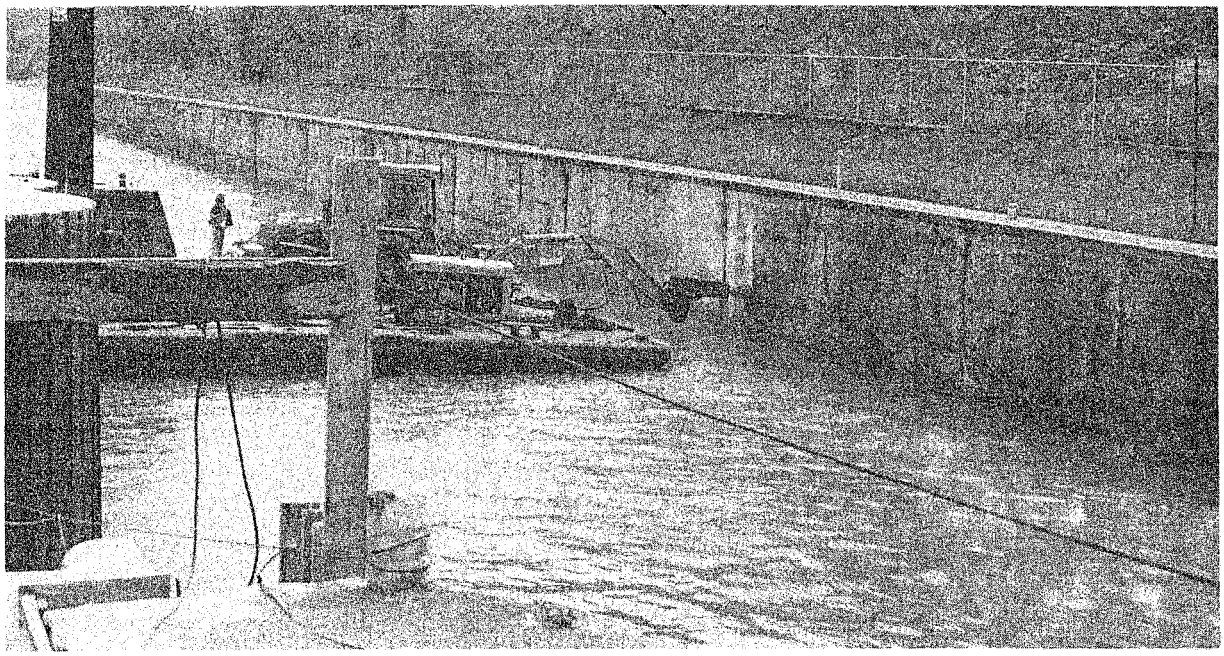


Figure 74. Bush hammer used to prepared lower guide wall for shotcreting, May 1980, Lock No. 3, Monongahela River

are driven by 15-hp Reliance electric motors. The hydraulic system is operated at 500 psi. The piping system is made up of pressure, return, and drain lines. All pipe is seamless black steel pipe, except pipe in the crossovers and embedded in concrete which is Type 304 stainless steel. The pressure lines are 2-in. Schedule 80. The return lines are 2-in. Schedule 40, and the drain lines are 1-in. Schedule 40.

162. The gate and valve operating machinery is controlled by double-solenoid operated, spring-centered, four-way valves. The filling and emptying valves for both chambers are controlled by 1-1/4-in. Racine valves, Model FD4-FSHS-110M-60, and the lock gates are controlled by 3/4-in. Racine valves, Model FD4-FSHS-106M-60. The valves are of submersible construction with oil immersed encapsulated solenoids. This joy stick activated valve provides infinitely variable gate speed from zero to the maximum. Speed variation is proportional to joy stick movement. Flow control valves are installed in the lock gate operating systems to control the speed of the gates. The valves are Racine Model FF2-EMSP-06E, pressure-compensated electrohydraulic type. The gate operating cylinders were manufactured by Hunt Valve, Salem, Ohio. The cylinders have a 10-in. bore and a 69-1/2-in. stroke. The new cylinders for the large chamber filling and emptying valves were also manufactured by Hunt Valve. They have a 3-in. bore and a 36-in. stroke.

163. The majority of the original cables used for distribution of power, lighting, and controls were either lead or parkway cable over 70 years old and badly deteriorated. All of the cables were replaced with General Electric Vulkene Power and Control Cables. New conduits and pullboxes were provided. New lock chamber cable crossings were installed through the crossovers. A secondary source of power is provided by a 150-kw, 187.5-Kva, three-phase, 60-Hz, Onan AC generator powered by an Allis Chalmers Model 1700 Mark II, six-cylinder diesel engine. The system is provided with an automatic transfer switch to provide uninterrupted power and automatic transfer between commercial and standby power.

164. The following operating systems were renewed:

a. Motor control center.

b. Traffic signal system.

c. Navigation light system automated using time clocks.

d. Valve-gate interlock system.

e. Lock wall lighting system using high-pressure sodium lamps.

f. Interior lighting systems for the middle and land wall buildings including new transformers.

165. Replacement of mechanical components under the rehabilitation contract was limited to the lock gate operating machinery. New sector gears, racks, and gate arms were installed for all lock gates except the upstream river wall gate. It was found that the sector for that gate was not the same size as the others. Consequently, a refurbished sector gear, arm, and rack were installed. The machinery for the filling and emptying valves in both chambers was not replaced by the contractor. It was originally contemplated that repairs to the land chamber culvert valves would be accomplished at a later date on an as-required basis after the major rehabilitation was completed. During the course of the work, it was found that there was considerable leakage through the filling and emptying valves. This additional work, including replacing filling and emptying valve machinery, valve blades, and stems was accomplished by the District Repair Party.

166. The major rehabilitation contract was completed in November 1980 at a total price of $12,452,000.

167. The second periodic inspection of the project was conducted in May 1981. The top surfaces of the land, middle, and river walls were reported to be in good condition, and the 1/2-in. sponge rubber type expansion joint material was performing well. The polymer mortar overlay of the filling flume deck slab had extensive cracks in some areas while other areas were practically free of cracks.

168. In the river lock chamber, the new vertical concrete facing on the river and middle walls contained two or three vertical hairline cracks and some minor areas of spalling at two or three monolith joints. One large area of gouged or spalled concrete at the waterline was located in the middle wall slightly upstream from the intermediate miter gate recess. In the land chamber, the land wall refacing concrete exhibited more extensive cracking (mostly vertical, but with some horizontal cracks as well) and more minor areas of spalling than the other walls. To a much lesser degree, the land chamber face of the middle wall also had a few cracks and spalled areas. A crack survey has been made by Design Branch personnel for record purposes, and the cracks will be checked periodically by project personnel. Overall, the concrete chambers in both lock chambers were in good condition.

169. The riverside of the river wall, the lower guide wall, all

existing gate recesses and various areas on the upper guide wall and middle wall were treated with a thin coating of shotcrete. Several small localized areas of the new shotcrete had been abraded and spalled, evidently by tows entering and leaving the locks. The shotcrete on the riverside of the river wall, an area not subject to tow impact, was in good condition with only minor shrinkage cracking. The performance and wearability of the new shotcrete coating in areas subjected to tow abrasion and impact was unsatisfactory, particularly where the shotcrete was applied in a very thin layer (less than 1/2 in.) over relatively smooth surfaces, and where exposure to impact and rubbing by tows is most severe.

170. In June 1982, isolated cracks were observed in the top surfaces of the lock walls (Figure 75). Most of the cracks are perpendicular to the face of the lock chamber. Although a few cracks were located on approximately 5-ft centers, the regularity noted at other projects was not observed. Rather, the cracks tend to start and terminate at reentrant angles and at joints in the corner protection armor. The thin polymer mortar overlay was debonded and buckled in several places (Figure 76). Failure of the shotcrete overlay appeared to be progressing (Figure 77).

171. In July 1985, additional failures of the shotcrete repairs were reported (Figure 78). Cracks in the top of the lock walls had been caulked with a material manufactured by Thoro System Products. The caulking was done meticulously by project personnel and is not unattractive (Figure 79).

Lock and Dam No. 1, Mississippi River

172. Lock and Dam No. 1 is located at Mississippi River mile 847.6 above the mouth of the Ohio River and between the cities of St. Paul and Minneapolis, Minnesota. The original structure was completed and placed in operation in 1917 and included a 152-ft long hydroplant adjacent to the left bank, a 574-ft crest-length, Ambursen-type dam surmounted by 2-ft high automatic release flashboards, eight sluiceways, and an 80 by 360-ft navigation lock. In 1929 the lock failed, cutting off all barge traffic to Minneapolis. To ensure against a future interruption to barge traffic, it was decided to build twin locks each 56 by 400 ft at this site (Figures 80 and 81). The first lock (river lock) was completed in 1930, and the second lock (land lock) was placed in operation in 1932.

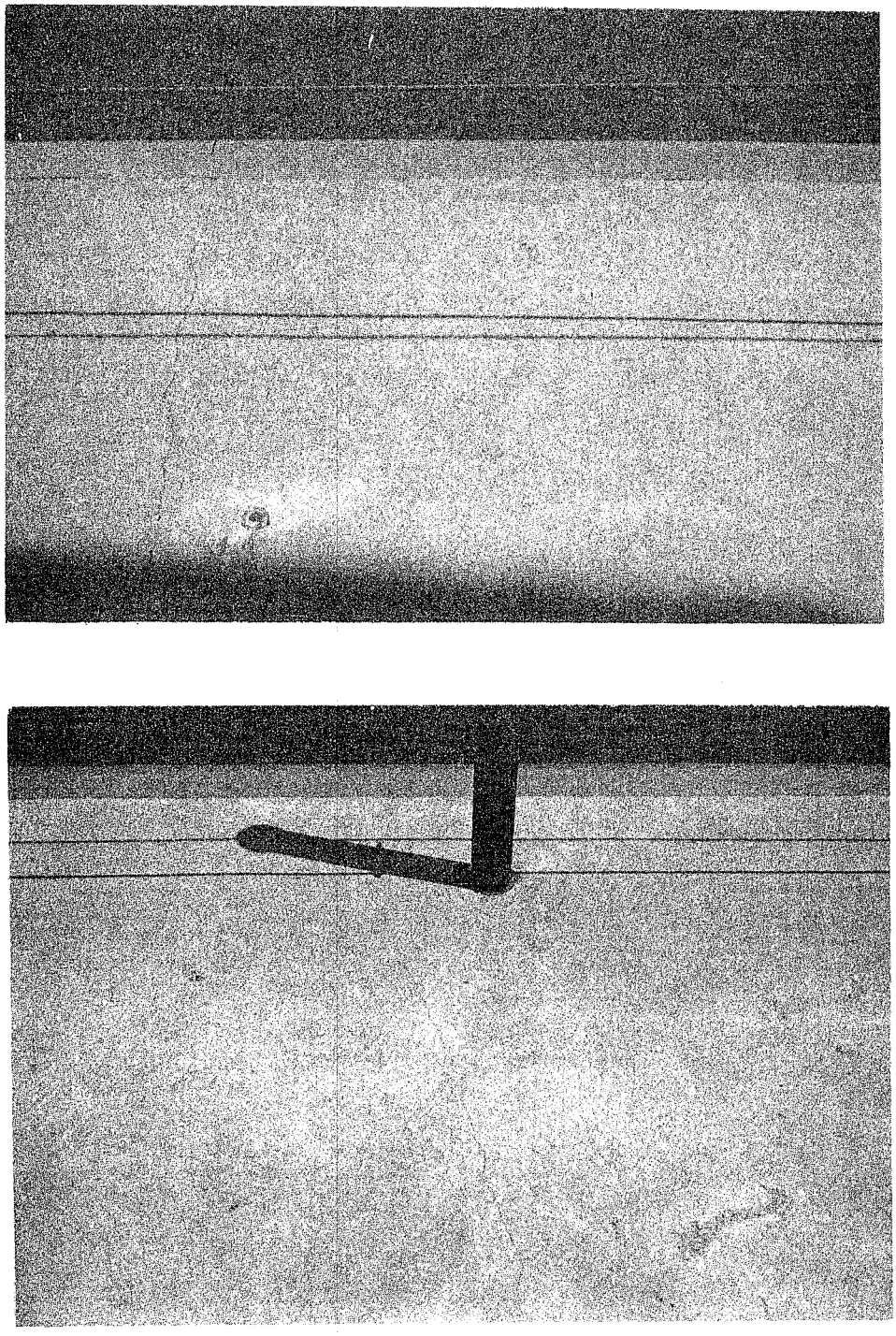


Figure 75. Typical cracks in tops of lock walls, June 1982, Lock No. 3, Monongahela River

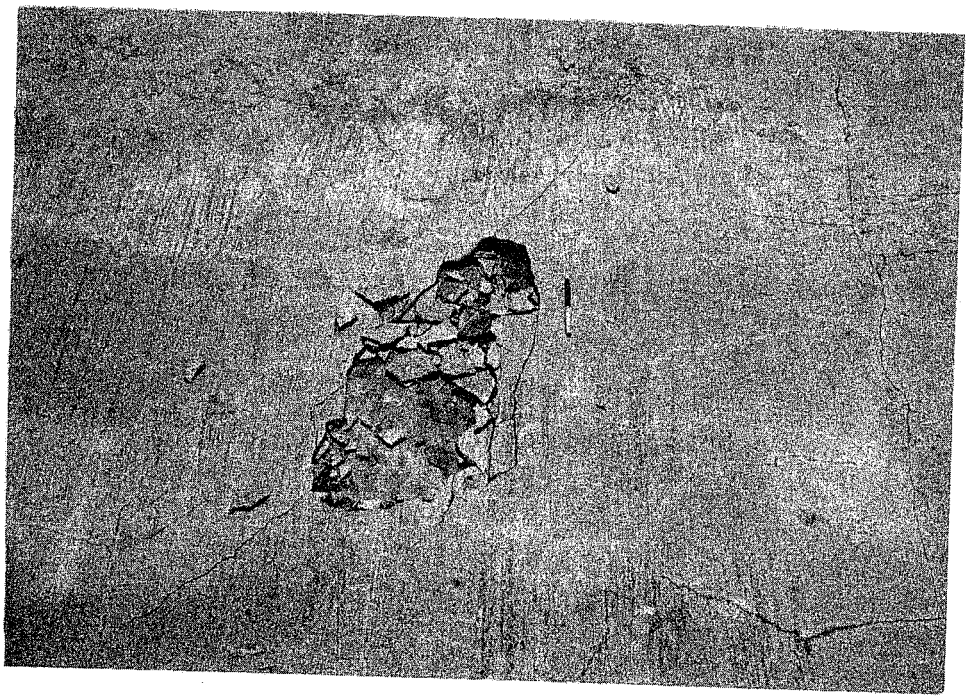


Figure 76. Polymer mortar overlay failure, June 1982, Lock No. 3, Monongahela River

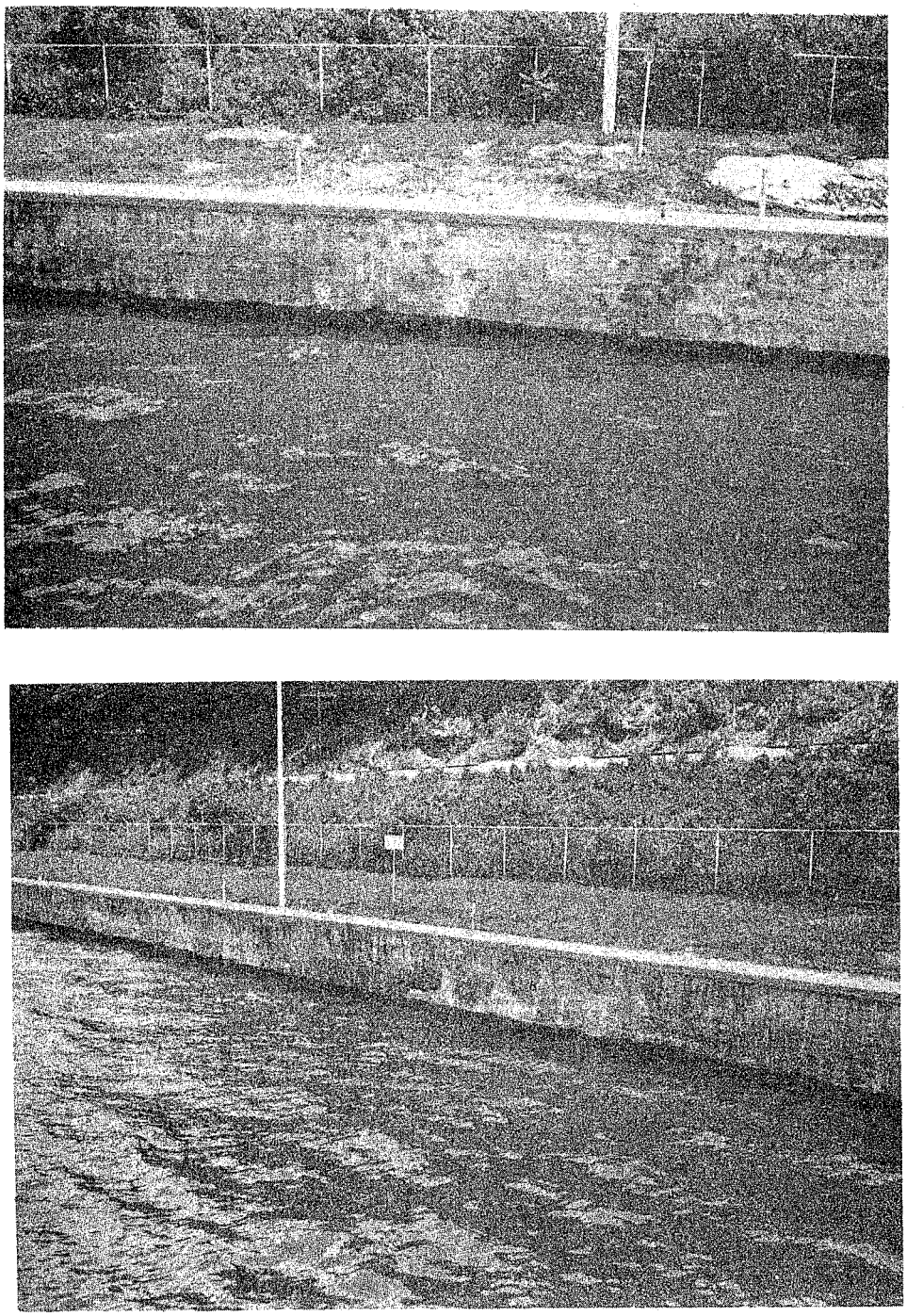


Figure 77. Typical spalling and abrasion of the shotcrete resurfacing on the lower guide wall, June 1982, Lock No. 3, Monongahela River

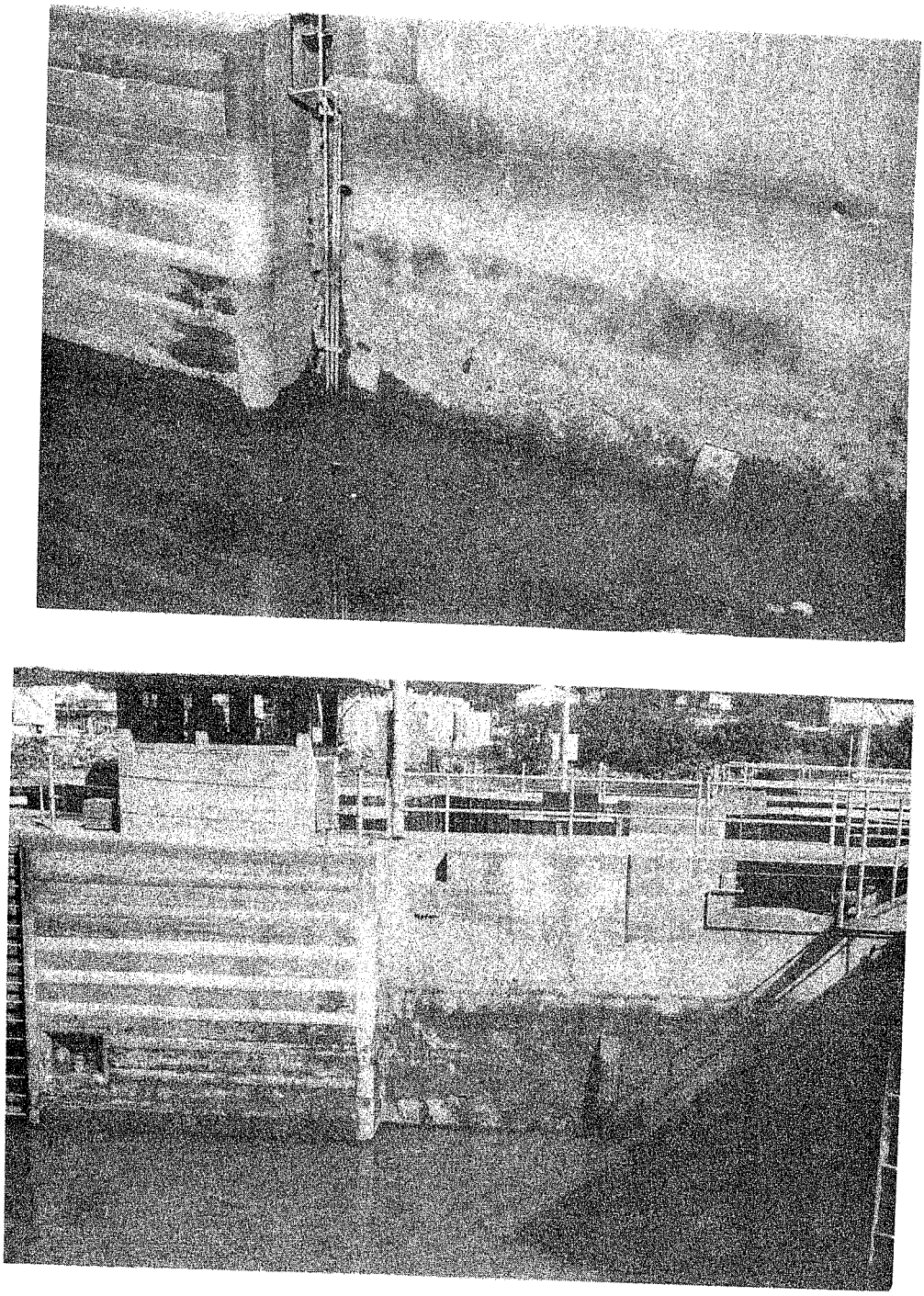


Figure 78. Spalling of shotcrete overlay in gate recess, July 1985, Lock No 3, Monongahela River

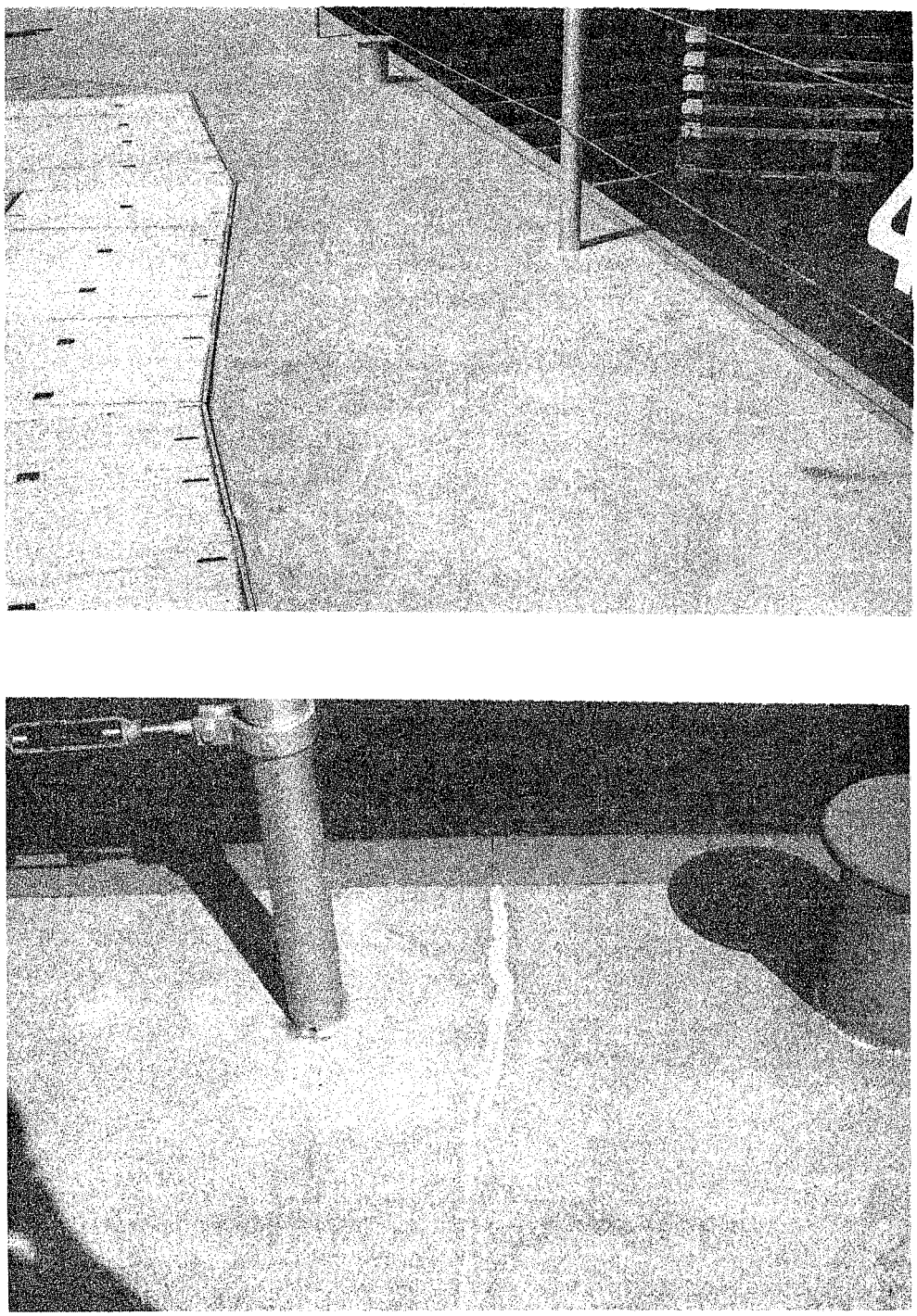


Figure 79. Caulked cracks in top of middle wall, July 1985, Lock No. 3, Monongahela River

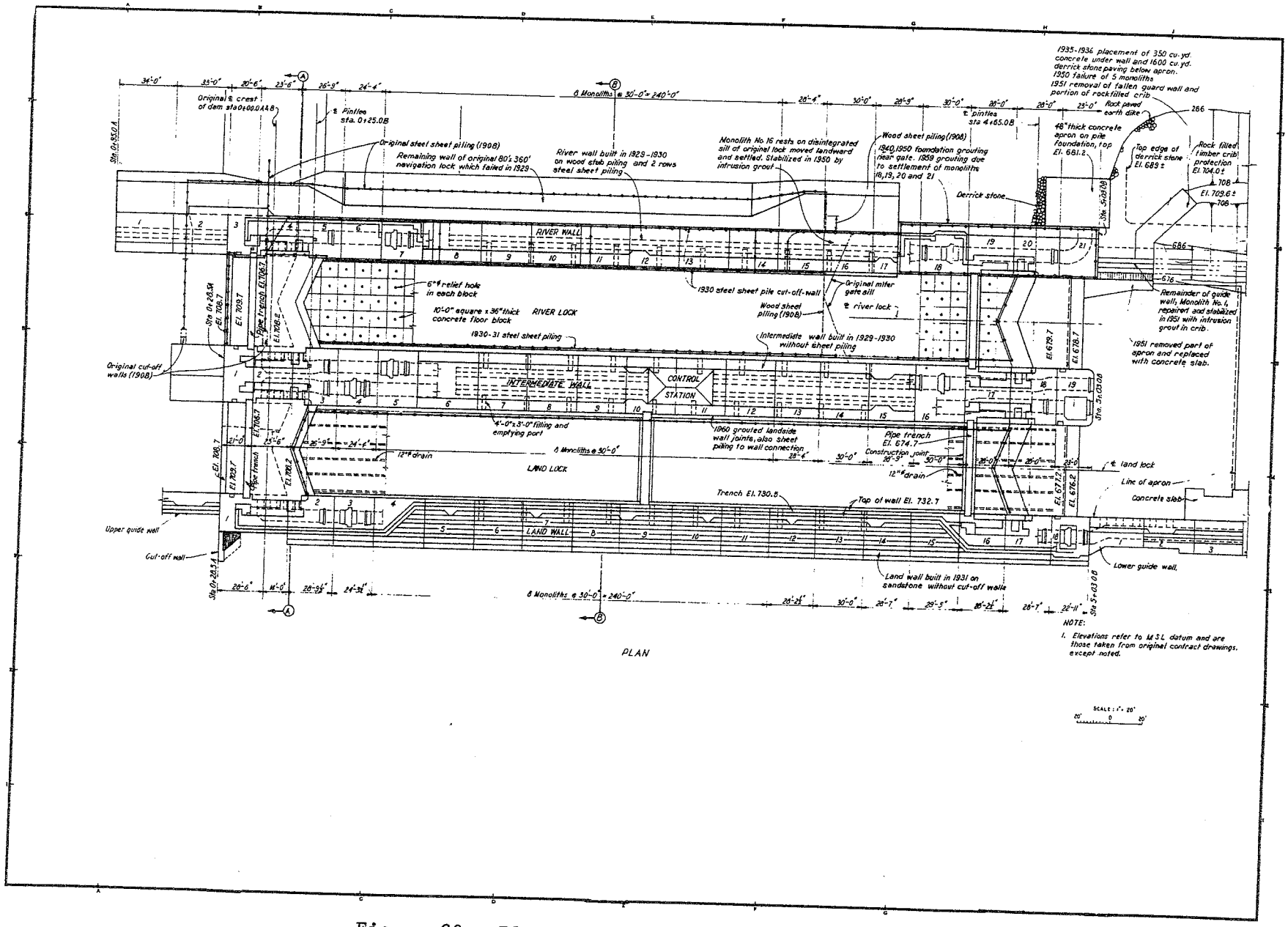


Figure 80. Plan, Lock No. 1, Mississippi River

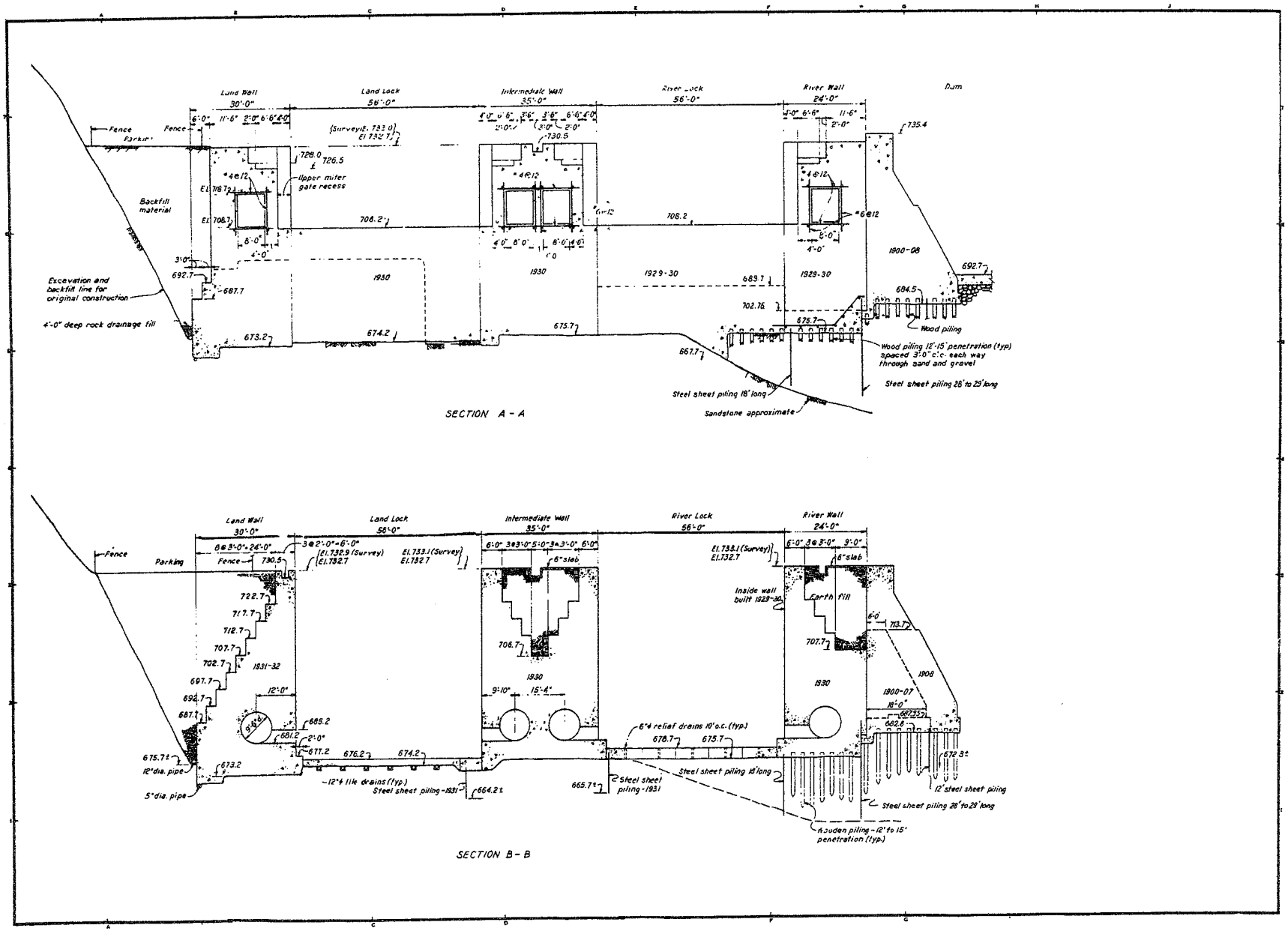


Figure 81. Typical sections, Lock No. 1, Mississippi River

173. The riverside lock was designed to provide a structure suitable for 9-ft draft navigation based on the design pool level for Lock and Dam No. 2, which was then under construction. However, because of probable seepage damages, interests in the South St. Paul stockyards area obtained a court order limiting the elevation to which the pool could be raised to 685.7 msl (mean sea level). Later, in 1934, the court approved the raising of the pool to elevation 687.2, 1.9 ft less than its designed height. As a result, there is a depth of only 7.5 ft over the lower sill at flat pool or about 8 ft at normal tailwater elevation. Hence, the lock has had little use except for an occasional locking of pleasure boats, empty barges, or shallow-draft towboats. The poor condition of the operating machinery and the lack of guide walls, making approach difficult, have also been factors in limiting use of the river lock. In building the river lock, the landward wall was constructed of adequate width, with two emptying and filling conduits, to serve as the intermediate wall of the twin locks when the second lock was constructed.

174. The landside lock was built in 1931-32 as a safeguard to maintain river traffic to and from Minneapolis. As a result of the failure of the original lock, Minneapolis was without barge line service for over a year. It was determined that a recurrence should be avoided if at all possible. The downstream sill of this lock has a top elevation of 677.2 providing a depth at flat lower pool of 10.0 ft or about 10.8 ft at normal tailwater elevation; hence, the land lock handles practically all commercial river traffic.

175. In March 1971, Periodic Inspection Report No. 1 was issued describing the conditions of the structures as found during a visual survey performed by an inspection team in August 1967. The report described the deteriorating condition of concrete, loss of fill material in joints, displacement of lock walls, questionable stability of several wall monoliths, and the unsatisfactory condition of operating machinery. It was concluded that problems continue to arise from unexpected sources and remedial measures would be necessary to maintain the lock in operation.

176. Detailed field and laboratory evaluations of the condition of the concrete at Lock and Dam No. 1 were conducted during the period from 1974 until the 1976-77 investigative dewatering. The purpose of the evaluations was to determine the structural adequacy of the concrete and the requirements for reconditioning so that the extended service life of the concrete would be compatible with the service life of other rehabilitated features. The need

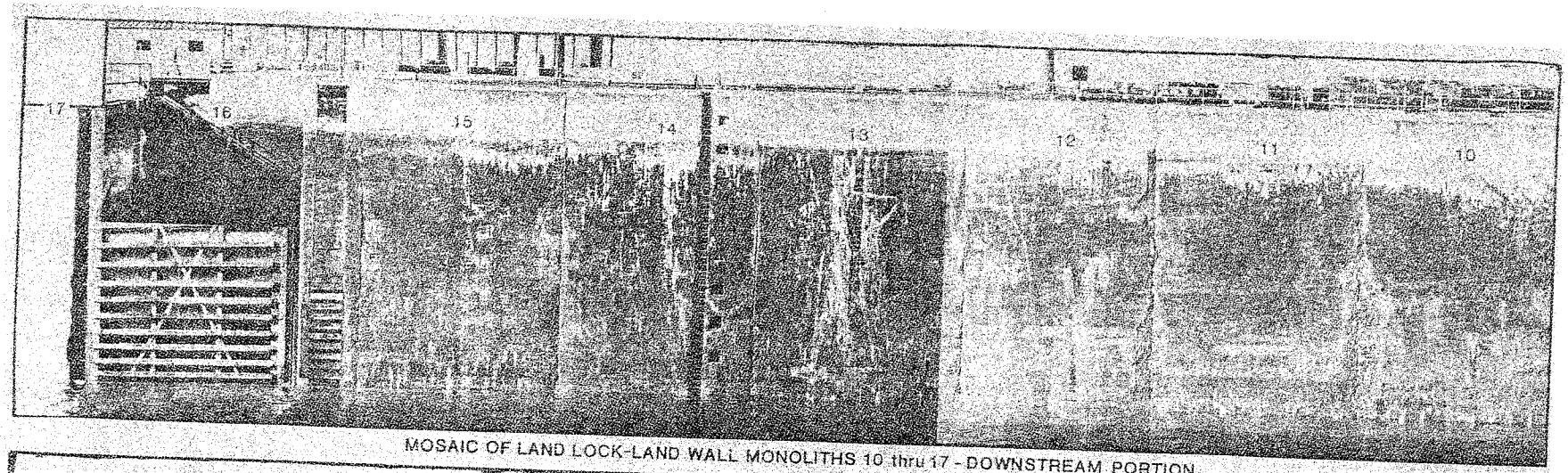
for such reconditioning was based on the degree of apparent surface deterioration, general lack of information on the quality of concrete materials and cement used in the original construction, and the results of laboratory compressive strength tests and petrographic examinations on concrete cores taken from various lock surfaces.

177. Concrete cores, NX-size, were obtained from vertical and horizontal drill holes at 36 locations within the locks. Petrographic examinations were made on samples of all cores. Water quality tests were made on water samples taken from beneath the land lock floor slab. Field investigations included crack surveys, pulse velocity tests on walls where cracking through the conduit crown was observed, dynaflect testing of the land lock floor slab, Schmidt impact hammer and Windsor probe tests, and measurement of strain across joints using electrical joint meters (St. Paul District 1978).

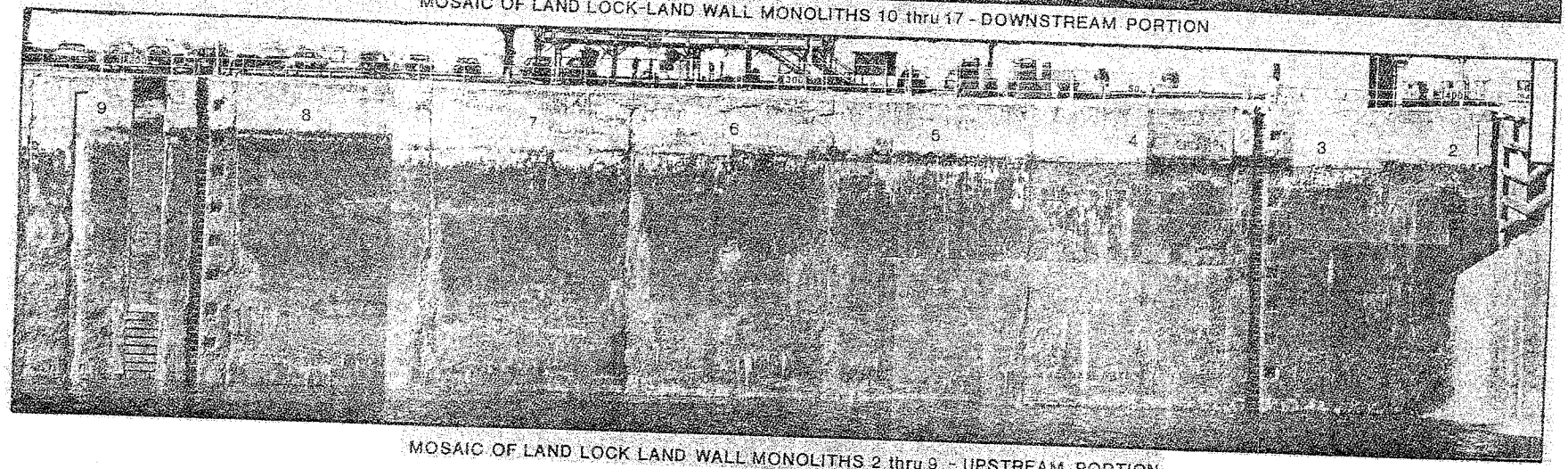
178. Based on the results of the concrete evaluations, it was concluded that the surface of the lock walls was undergoing a slow and steady deterioration from exposure to cycles of freezing and thawing. Typical concrete surface conditions are shown in Figures 82 and 83. The degree of deterioration was greater than was visually evident since there were wall surfaces which appeared sound but were deteriorated behind the surface. Such deterioration was generally characterized by laminar cracking parallel to the wall surface. The susceptibility of the lock concrete to deterioration from cycles of freezing and thawing was attributed to the degree of saturation, the lack of air entrainment, and the presence of porous chert, argillites, and microcracking as a consequence of the alkali-silica reaction or disruption of highly porous particles of coarse aggregate in the near surface zone.

179. The concrete deterioration was not confined to one area but was scattered over the entire lock surface. The surfaces most seriously affected were the top and edges of the walls and slabs, especially those above tailwater. Petrographic examinations of the concrete cores indicated wide variations in the depth of deterioration varying from 0 to 10.5 in. in the land lock and 0 to 15.7 in. in the river lock. Concrete in the walls below tailwater appeared adequately sound. Concrete in the lock floor slabs was in good condition with only superficial spalling at corners and edges. Water quality tests indicated that chemical attack on the concrete by river or groundwater was negligible at the time.

180. It was concluded (St. Paul District 1978) that some areas of the



MOSAIC OF LAND LOCK-LAND WALL MONOLITHS 10 thru 17 - DOWNSTREAM PORTION



MOSAIC OF LAND LOCK LAND WALL MONOLITHS 2 thru 9 - UPSTREAM PORTION

Figure 82. General condition of the land wall, land lock, prior to rehabilitation, Lock No. 1, Mississippi River

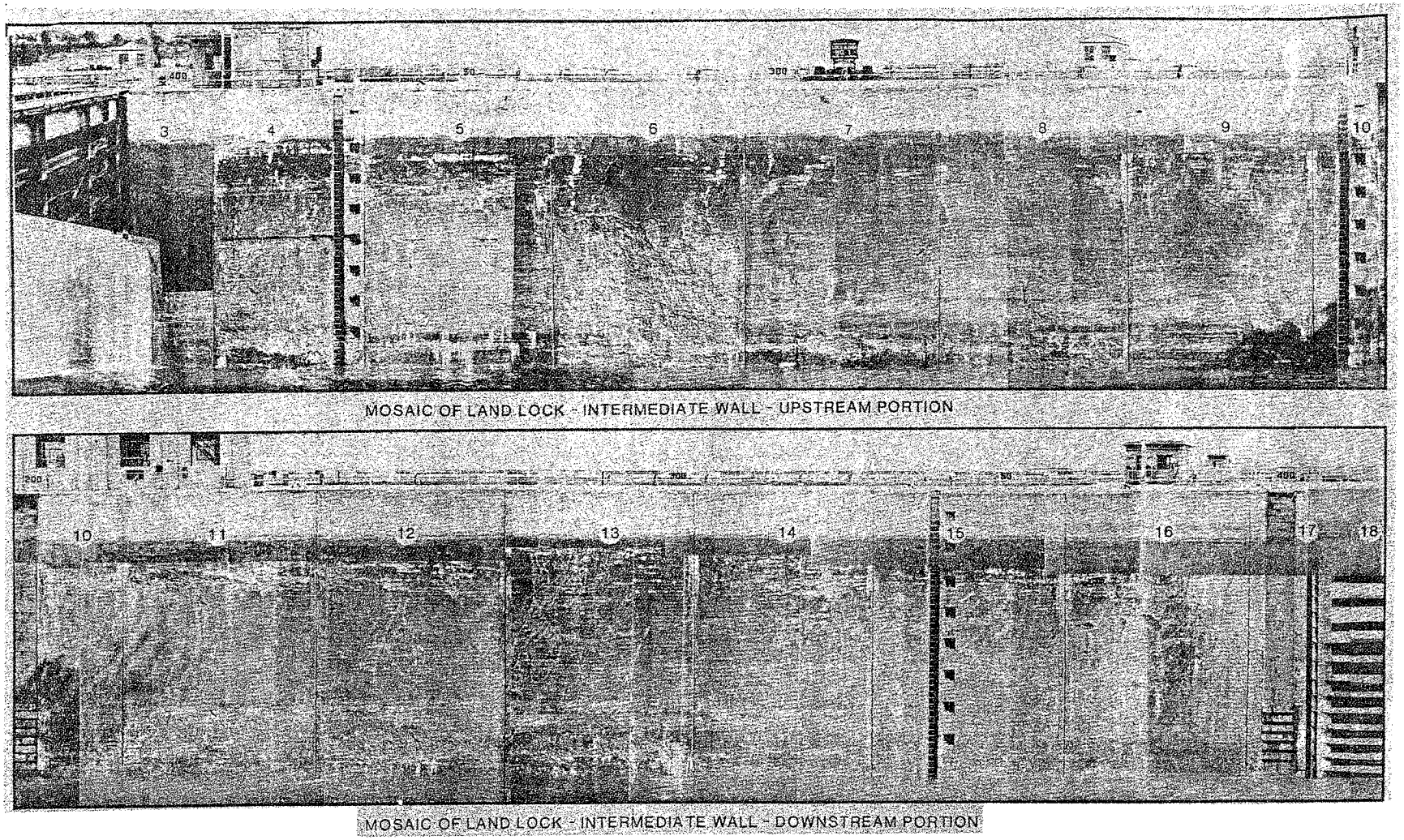


Figure 83. General condition of the intermediate wall, land lock, prior to rehabilitation, Lock No. 1, Mississippi River

lock walls would require extensive repair while other areas would need no restoration. However, determining the actual deteriorated areas would be difficult because the microcracking and laminar cracking are not always apparent. Repairing only the visibly deteriorated areas would result in an unsightly "patch up" job which would give a mottled appearance and would exclude many areas of necessary repair. Deterioration would continue in these unrestored areas and would require continuous maintenance. Therefore, for proper and effective restoration, the exterior concrete should be removed from the entire wall surface to below tailwater level or approximate elevation 686 msl and should be replaced with a durable protection material. The advantages of this method of repair are that: (a) it would ensure repair of all deteriorated areas including the nonvisibly distressed areas; (b) it would protect interior concrete from further deterioration; (c) it would extend the service life of the structure; (d) it would reduce or eliminate the need for extensive future maintenance and repairs which will become more costly; and (e) it would be more aesthetically pleasing.

181. A longitudinal crack running approximately the full length of each filling and emptying conduit was reported during a dewatered inspection of the lock during the winter of 1976-77. The crack was located near the crown of each conduit and could be traced across monolith joints. Since the cracking could present structural stability problems, 12 Carlson joint meters were installed in the conduits to measure crack movement during filling and emptying operations. Only two of the joint meters indicated movement. The amount of movement registered was 0.05 in. during filling of the land lock with the river lock held at tailwater level. Within 2 hr of the filling of both locks the crack returned to its initial position.

182. Ultrasonic pulse velocity measurements were made on the intermediate wall to determine if cracks in the crown of the conduits extended upward to the wall backfill (Figure 81). Personnel from WES made velocity measurements at 50 locations along the wall. Pulse velocities at 22 locations were within the expected range for sound concrete, lower velocities were measured at 17 locations, and no readable signal was obtained at the remaining locations. The low velocities and lack of readable signals were attributed, in part, to surface deterioration and near surface laminar cracking. Based on these tests, it was generally concluded that cracks in the crown of the conduits had not propagated to the wall backfill.

183. It was estimated that there were approximately 1,900 linear feet of vertical cracks in the monolith walls and separations along horizontal construction joints which would require repair. The majority of the vertical cracks were located in the river wall of the river lock; however cracking was evident in all walls. Some cracks were seeping water while others were stained from leaching. Unless the cracks were repaired, they would likely reflect through any replacement concrete and cause a future maintenance problem. In general, the joints appeared tight with no visible evidence of seepage; however some isolated vertical and horizontal joints were seeping water.

184. Based on the field and laboratory evaluations of concrete condition, the following recommendations for reconditioning the lock concrete surfaces were made (St. Paul District 1978):

a. Remove concrete on all exposed wall surfaces of the land lock to a minimum depth of 15 in., or to sound concrete in deteriorated areas. Removal will be by the preshearing technique using controlled blasting. New concrete will be placed to the original lines.

b. Epoxy grout cracks and leaky horizontal construction joints to eliminate reflective cracking through the new concrete overlay.

c. Replace the tops of all lock walls, including edges, to existing line and grade by replacement with conventional concrete.

d. Seal vertical joints in walls and slabs by application of sealing compound.

e. Place protective wall armor rubbing strips in selected areas such as along the upper and lower guide walls and the edges of recesses in the land lock chamber.

185. In addition to the concrete deterioration, three other significant problems made rehabilitation of the lock necessary (Plump 1986). First, an excessive amount of turbulence occurred in the lock chamber during filling and downstream of the lock during emptying. This turbulence presented difficulties even for commercial traffic using the lock and was a significant hazard for recreational boats. Second, the land wall, lower guidewall, and dam apron did not meet current stability criteria. Third, the operating machinery, electrical distribution system, and other similar facilities were outdated, in constant need of repair, expensive to maintain, and very inefficient.

186. Solutions to the lock turbulence problems were developed through extensive model testing at WES. These model tests indicated that lowering the lock culvert inverts and changing from a circular shaped culvert to a

rectangular shape would eliminate air entrapment in the culvert crown, a major reason for unacceptable turbulence in the lock chamber. Also, plans for new intake manifolds, discharge laterals, and filling and emptying valves and ports were developed based on model test results.

187. Because of the importance of the lock to the Twin Cities' economy, the rehabilitation had to be completed with minimal disruption to river traffic. Since the Mississippi River is closed to navigation for 3 months during the winter, the rehabilitation was scheduled to take advantage of this closing. Two 5-month-long winter dewaterings were planned for concrete removal and replacement required for lock wall restoration and hydraulic modifications including installation of the operating machinery. Other rehabilitation features could be constructed during the navigation season with the lock in operation.

188. Plans for rehabilitation of the lock required removal of extensive amounts of existing concrete. The only feasible means to remove extensive amounts of concrete in a minimum time frame was the use of controlled blasting. Several rehabilitation projects had used blasting for lock wall resurfacing. However, the use of explosives to create conduits within a solid mass of concrete without damaging the remaining concrete required advanced, state-of-the-art controlled blasting. The objective was to remove portions of concrete monoliths in a way that would preclude structural damage to the remaining concrete and to the foundation, which is a loosely cemented St. Peter sandstone. In order to develop a blasting plan that fulfilled this objective, the St. Paul District contracted with Woodward-Clyde Consultants of Chicago, Illinois, to prepare a test-blasting program which was executed during the 1978-1979 winter dewatering. The program was divided into three phases: development of a test-blasting program, execution of that test program, and preparation of a final report and plans and specifications for the concrete demolition (Plump 1982).

189. The initial step of the first phase was the development of damage control criteria. These criteria were to be used as guidelines in designing the test-blasting program. The first several steps in the execution of the test-blasting program would be designed to raise or lower the criteria level; but since the entire program had to be designed before any field data could be generated, these theoretical criteria would be used. Assume an ultimate

tensile stress of 300 psi in concrete and that stress and particle velocity are related according to the formula:

$$\sigma = \rho v c$$

where

σ = stress, psi

ρ = mass density, lb-sec^2/in.^{4}

v = particle velocity, in./sec

c = pulse velocity, in./sec

Then, the theoretical particle velocity causing damage by tensile cracking is 9 in./sec. Factors for the test program such as hole patterns, charge size, and detonation sequencing were designed based upon the damage control criteria.

190. Developing a testing sequence to determine the actual particle velocity causing structural damage and the effect of various charge shapes was the second step. A sequence of three test holes which would be taken to failure (complete blowout) was designed. The first hole would be loaded with concentrated charges, the second with line charges, and the third with a combination line and concentrated charges.

191. Next, the instrumentation for monitoring the test blasts was designed. Instrumentation would be installed within 4 in. of the charge, resulting in frequencies much higher (on the order of 10,000 Hz) than those which seismographs can monitor. Accelerometers are extremely accurate but very expensive and would, therefore, greatly reduce the number and location of instruments which could be used. Strain gages are inexpensive and compatible with the frequencies and wave lengths which test blasts would generate. When amplified and recorded on magnetic tape, the strains could be played back slowly and evaluated. Since strain is related to particle velocity $\varepsilon = v/c$ where ε = strain, in./in., strain gages were chosen as the principal means for evaluating the test blasts and damage. The gages were backed up with the use of accelerometers, seismographs, piezometers, high-speed photography, concrete surveys, level measurements, and extensometers.

192. The final step of the first phase was to design a procedure for testing actual full-scale production shots to be used in the removal of concrete during rehabilitation. Even though the design of the production shots

would change somewhat as a result of the initial test blasts and subsequent modification of the damage control criteria, it was necessary to have an estimate of these shots in advance to ensure that the labor, equipment, and materials would be available to carry them out. The execution of the test-blasting program immediately followed the dewatering of the locks in December 1978. Horizontal 4-in.-diam concrete cores were taken in 28 different locations. These cores were then cut in half along the horizontals axis, and five pairs of strain gages were mounted on one half of each of the 28 cores. Each pair of strain gages consisted of one primary gage and one backup gage. An accelerometer was mounted on the end of each of the 28 half cores, and the half cores containing this instrumentation were then grouted back into their respective holes. Twelve extensometers were installed in the concrete monoliths above and below the test blast area, and the other instrumentation was also prepared. The initial test blasts were then conducted to determine the strain and particle velocity at which damage to the concrete would occur. This testing took approximately 1-1/2 weeks, and the results are summarized in the following tabulation:

Effects	Strain millionths	Particle Velocity in./sec
Theoretical static failure in tension	60	9
Spalling of freshly set grout	700	100
Spalling of weathered surface concrete	1,300	200
Cracks develop extending from shot holes	2,400	375
Mass concrete blown out	3,800	600

The results of this testing demonstrated that strains (or particle velocities) in excess of simple theoretical values could be used to safely design a program of concrete removal using explosives without damage to remaining concrete. It also was determined, following plotting and analysis of the data, that the strain levels causing damage were predictable and that simple tests (much less extensive than these) could be designed to predict strain levels at which damage occurs.

193. Two methods of production-type blasting were tested for removal of concrete to lower the circular filling and emptying conduits. The first was

the detonation of a series of holes drilled radially along the bottom half of the conduit. The second method was the detonation of approximately forty 6-ft holes drilled horizontally along the axis of the conduit and below it. This method proved more feasible for several reasons:

a. The vibration effect on the foundation below the conduit was negligible.

b. The concrete remaining after detonation was structurally sound and required no cleanup or handwork prior to placing new concrete against it.

c. Muck was confined to a small area where it was easily cleaned up following detonation.

d. Charge weights could be reduced substantially since the majority of the concrete was being broken in tension rather than in shear.

The techniques which were tested proved that selective concrete removal by explosives was not only feasible from a technical point of view but also economically superior to other types of nonblasting techniques. The St. Paul District published a report describing in detail the procedures and results of this test-blasting program (St. Paul District 1982). This publication is available to other Districts and should be of value where selective concrete removal is required within a limited time frame.

194. The plans for rehabilitation of Lock and Dam No. 1 were extensive. The land lock, which carries virtually all of the river traffic, received the most rehabilitation attention. The river lock is rarely used and was only partially rehabilitated. Lock rehabilitation required a new filling and emptying system, replacement of deteriorated concrete, installation of post-tensioned anchors through the lock walls for stability, plus complete replacement of operating machinery, and power supply and control systems. The apron slab of the overflow spillway dam also required anchoring with posttensioned anchors and the replacement of some deteriorated concrete.

195. A few months in advance of advertising for rehabilitation bids, it became clear that the design documents would not be completed in time for a fall 1979 advertising. However, rather than postpone advertising, which would have required delaying construction a full year, a decision was made to use staged construction (St. Paul District 1983). Stages of construction were as follows:

Stage No.	Description
1	Hydraulic modifications, posttensioning of intermediate and land lock walls, and wall resurfacing.
2	Buildings, pedestrian bridges, river wall improvements, and external utility systems.
3	Water and sanitary sewer systems.
4	Rehabilitation of dam and lower guide wall.
5A	Safety walls.
5B	Access road.
5C	Security fencing.

The Corps took responsibility for the acquisition of items with long lead times. Three separate supply contracts were awarded for sheetpiling, fabricated metal items such as the tainter valves, and the miter gate and tainter valve operating machinery. This arrangement ensured that these items were on hand and available to the contractor as needed during construction.

196. The Stage I contract, covering approximately 65 percent of the total project work, was awarded to Al Johnson Construction Company, Minneapolis, Minnesota, in October 1979 for $19,629,738. During the first winter dewatering, 1979-80, the tainter valve monoliths were rebuilt and the filling and emptying conduits modified.

197. Controlled blasting was used to carefully remove the concrete in the culverts, to replace the face of the lock walls, and to remove monoliths of concrete as large as 30 ft wide, 30 ft deep, and 60 ft high necessary for the modifications to the filling and emptying system. To ensure the safety of the structure during construction, the St. Paul District decided to use a method specification for the blasting. This technique removed all responsibility from the contractor for the blasting results, because the plans and specifications detailed the hole sizes and spacings, explosives and amounts to be used, and the sequential firing arrangements. The District obtained the same blasting expert who managed the test program to prepare the design and to supervise inspection of the blasting related construction activities. Although results were not always as anticipated and some modifications were required, in no instance did the blasting ever cause any unanticipated damage.

198. Modifications to the existing filling and emptying culverts involved lowering the inverts by 2.5 ft and changing from a circular shape (9.5-ft diameter) to a rectangular shape 9.5 ft wide by 7.5 ft high

(Figure 84). The blasting experts enlarged one of the ports through the lock walls to gain access to the culverts, then blasted a slot in the invert of the culverts to permit horizontal holes to be drilled parallel to the culvert walls along the final boundary of the modified culvert. Small blasting charges and timed delays allowed removal of the required concrete (Figure 85) without damage to the underlying sandstone foundation and without fracturing the remaining concrete between the culvert and the lock chamber. After blasting, some areas of the remaining concrete were less than 2.5 ft thick. A total of 14,000 cu yd of concrete was removed throughout the lock, mostly by controlled demolition. The use of controlled blasting on this project required an advance in the state of the art, and an extensive amount of testing was necessary. The results were excellent, and blasting was the single most important aspect in the successful completion of the project (Passage and Plump 1984).

199. When the concrete had been blasted and removed from the culverts, the new culverts had to be constructed quickly. The conventional approach

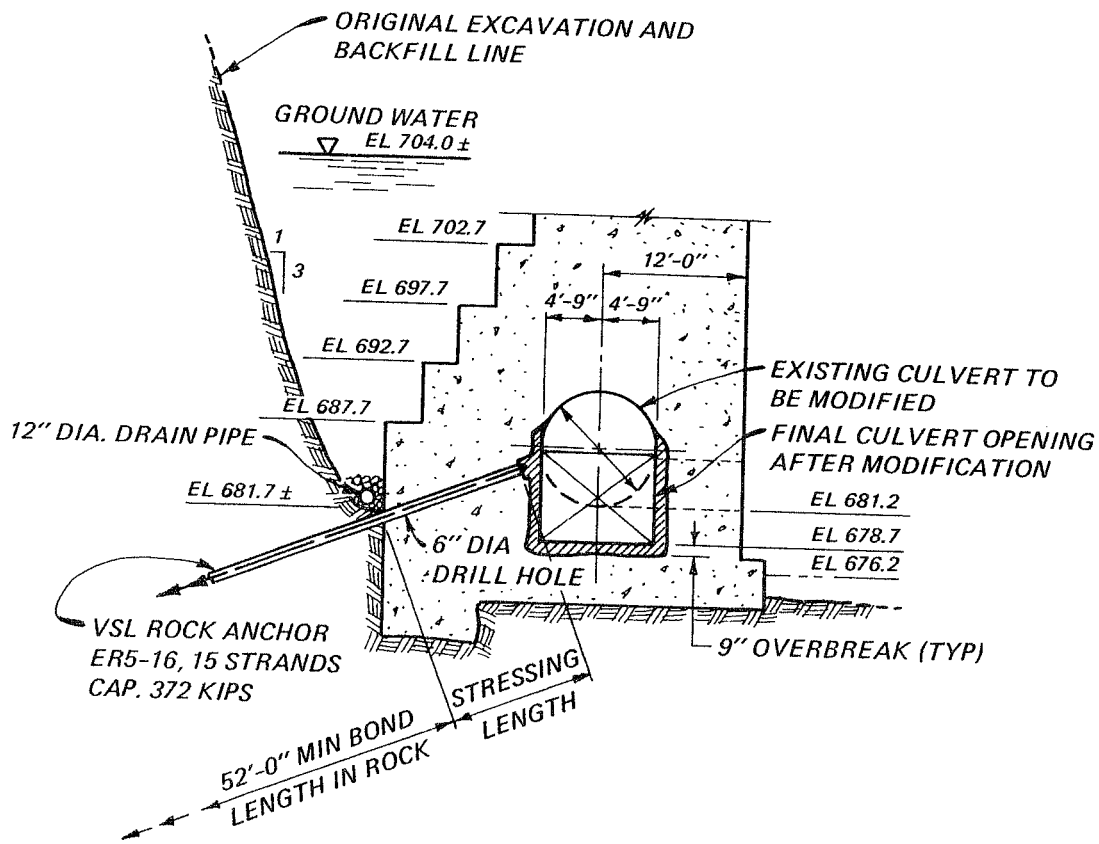


Figure 84. Typical section through landwall culvert, Lock No. 1, Mississippi River

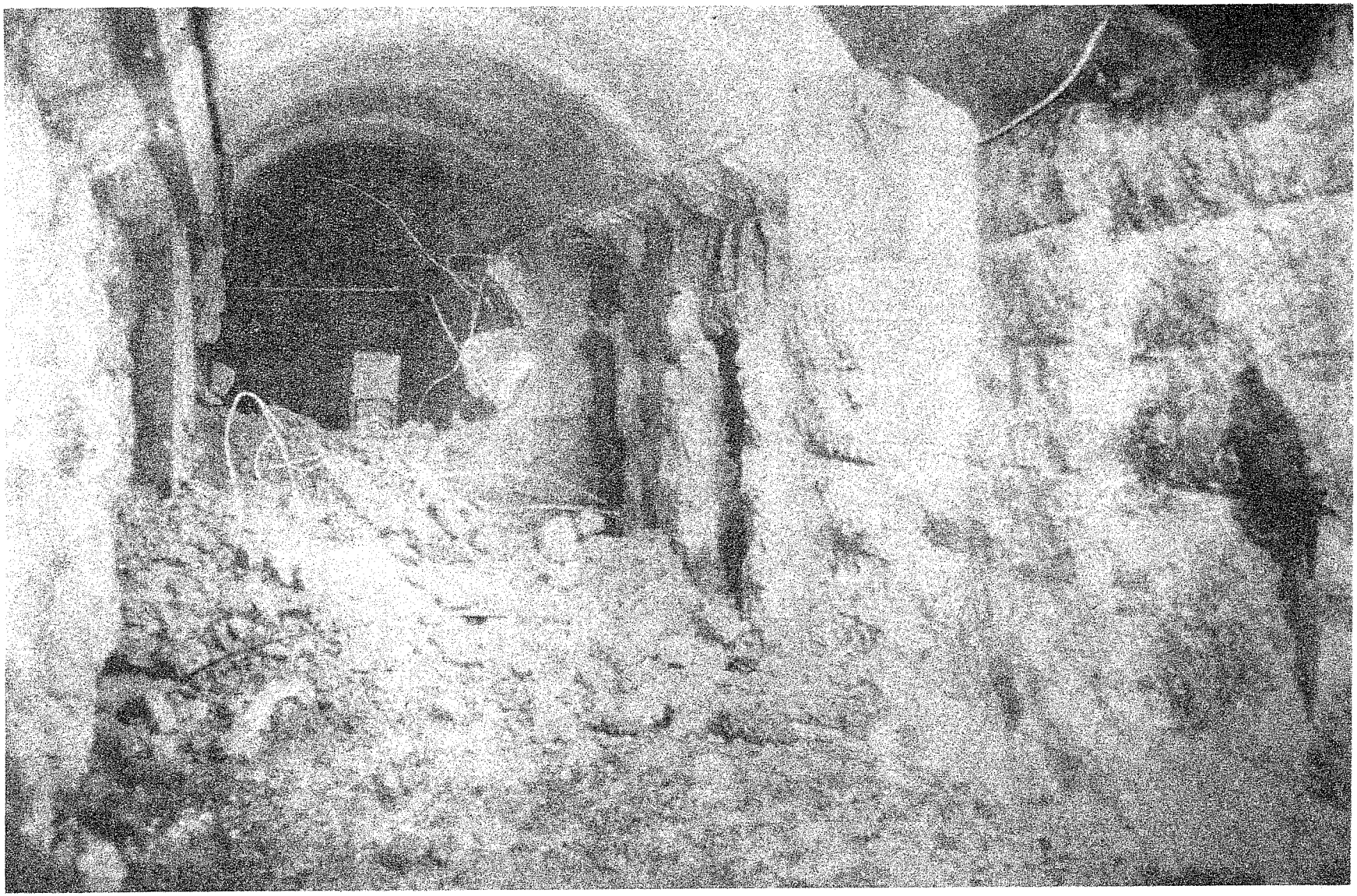


Figure 85. Concrete removal using explosives, Lock No. 1, Mississippi River

would have been to drill and set dowel bars, place concrete reinforcing and forms, then wait for the fresh concrete to set before removing the forms. Instead, permanent steel liner segments used as forms for the concrete speeded the process. They were positioned along a set of steel tracks and bolted together, and the space behind the liner was pumped full of concrete. This method saved weeks of valuable construction time during the 5-month winter construction period.

200. The 60-ft-high lock walls did not meet acceptable criteria for stability under dewatered lock conditions. Posttensioned anchors through the lock culverts would correct this problem, and 43 were installed in the land wall over a period of two years. Posttensioned anchors were also installed horizontally through the intermediate wall to negate the crack that propagated upward from near the crown of the filling and emptying culverts. These anchors were necessary to allow for concrete removal by blasting on the intermediate wall. The tight working conditions in the culverts and limited time frame required that the posttensioned anchors be drilled and installed, but unstressed, during the investigative dewatering of the locks. The anchor strands were left recessed in the culvert walls and covered with concrete for protection. During the following winter's modification of the culvert geometry, the anchor strands were reexposed by the blasting (Figure 84) and then tensioned.

201. The existing vertical lift gates in the filling and emptying system required extensive maintenance every five years, and spare parts were no longer manufactured. Therefore, they were replaced with reverse tainter gates which have a proven history of reliability and low maintenance. Since the existing blockouts in the value monoliths would not accept the tainter valve machinery, adjacent lock wall monoliths had to be demolished and reconstructed to accommodate the new, more efficient tainter gates. Generally, concrete was placed in 5-ft lifts during reconstruction of the valve monoliths (Figure 86).

202. During May 1980, a trial lock wall resurfacing was conducted on monolith L-5. Prior to resurfacing, existing concrete was removed from the chamber face to an average depth of 18-in. Reinforcing consisted of No. 6 bars on 12-in. centers each way anchored with No. 8 dowels 3 ft on centers (Figure 86). The dowels were embedded 18 in. into the existing concrete with polyester resin cartridges. The area to be resurfaced was formed and placed in two lifts 23 ft high and 30 ft wide. A 3/4-in. thick plywood faced form

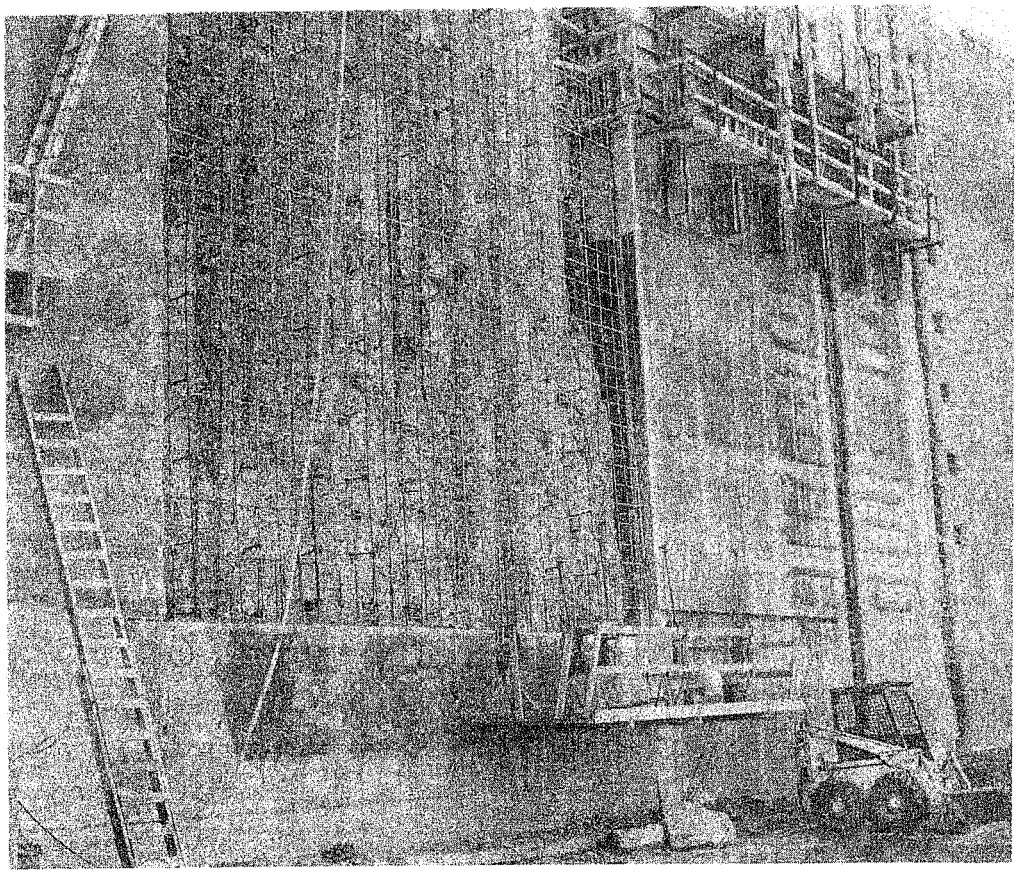


Figure 86. Trial resurfacing monolith L-5 prior to forming (left) and tainter valve monolith L-4 reconstruction, 9 May 1980, Lock No. 1, Mississippi River

was used for both lifts. The single section form (Figure 87) with structural aluminum bracing was held in position with 1-1/4-in.-diam anchor bolts spaced 6 ft on centers. The anchor bolts were embedded 24 in. into the existing concrete with polyester resin cartridges. The existing concrete was washed with a water blaster (7,000-psi pressure) the evening before each placement.

203. Trial lifts A and B were placed on 17 and 20 May, respectively, using a concrete pump with a 6-in.-diam line and a 4-in.-diam elephant trunk. Approximately 54 and 46 cu yd of concrete were placed in lifts A and B, respectively. The concrete was placed at an average rate of 5.7 ft/hr for both lifts. Air-powered internal vibrators (2-in. diameter) were used to consolidate the concrete. A concrete mixture proportioned with 1-1/2-in. maximum size aggregate (MSA) to have a 28-day compressive strength of 3,000 psi was originally intended for the lock wall resurfacing. However, since other placements scheduled for 17 and 20 May required concrete with 4,000-psi

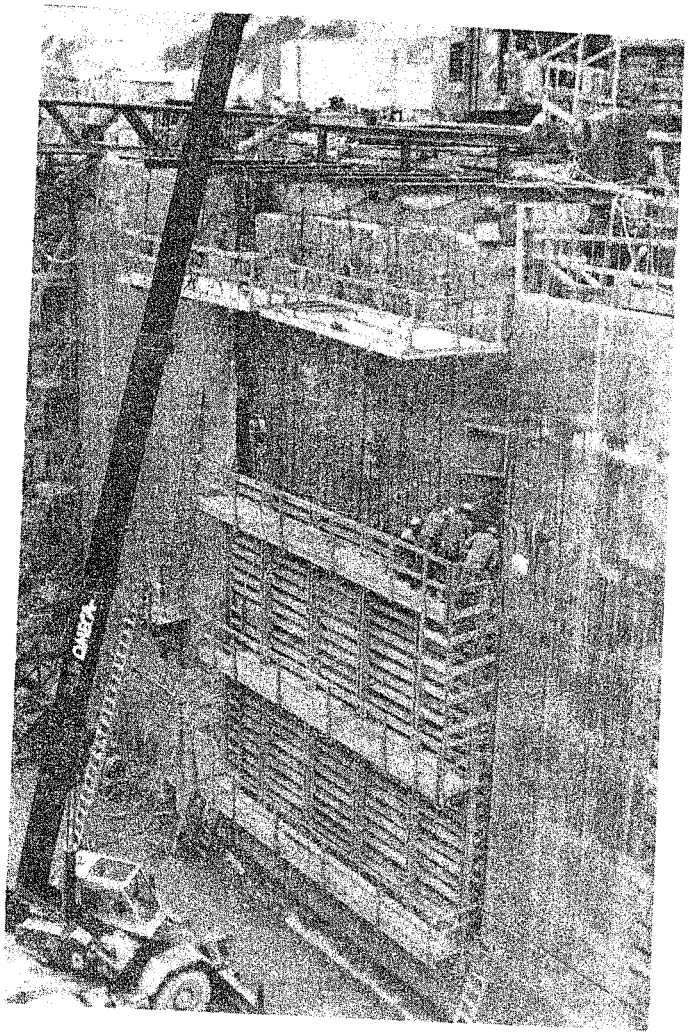


Figure 87. Form work in place for Lift A of trial resurfacing, 12 May 1980, Lock No. 1, Mississippi River

compressive strength at 28 days age, the higher strength mixture was also used for the resurfacing placements. Mixture proportions, based on a 1-cu yd batch, were as follows:

Mixture No. 560-6PA6 (Pump)

Material	Weight
Portland cement, type I	560 lb
Fine aggregate	1,345 lb
Coarse aggregate (3/4-in. MSA)	1,730 lb
Water	265 lb
Water-reducing admixture	22.4 oz
Air-entraining admixture	4.5 oz

Average properties of the fresh concrete were as follows:

Property	Lift A	Lift B
Slump, in.	2-1/4	3
Air content, %	4.0	5.8
Temperature, °F	64	64

Ambient temperatures measured at the site during concrete placement averaged 54 and 79°F for lifts A and B, respectively.

204. Forms for lifts A and B were removed between 0830 and 1030 hr on 19 and 21 May, respectively. No measure were taken to cool the forms between the time of installation and removal. A nonpigmented, wax base curing compound was sprayed on both lifts approximately 8 hr after form removal. Only the top portions of each lift were water cured. Neither lift was shaded from the sun before or after curing compound application. Weather conditions during the curing period were hot and sunny with low humidity and moderate winds (Figure 88).

205. On 23 May, the lock chamber was flooded to low pool elevation; on 26 May, the water level was raised to the upper pool elevation. The water temperature was approximately 50 to 55°F. On 27 May, extensive cracking was observed in both lifts (Figure 89). The pattern cracks were generally spaced at 2 to 4 ft intervals. Crack widths at the surface appeared to be essentially constant, less than 0.01 in. wide. It could not be determined when cracking actually occurred, but the cracks were not observed until the lock chamber was placed into operation. Typically, cracks are much more visible after a concrete surface has been wetted since the cracks will retain moisture much longer than the surrounding concrete surface. Similarly, concrete cracking is usually more apparent following application of nonpigmented curing compound.

206. Concrete cores were obtained from each lift as part of the District's investigation into the possible causes of the cracking. Two pairs of 4-in.-diam cores were obtained from each lift. In each case, one core was drilled horizontally along a crack, and a second core was drilled through uncracked concrete in the same vicinity. All cores were drilled to a depth of 4 in. beyond the interface between the existing and replacement concretes. Seven of the eight cores were retrieved with the interface intact. An

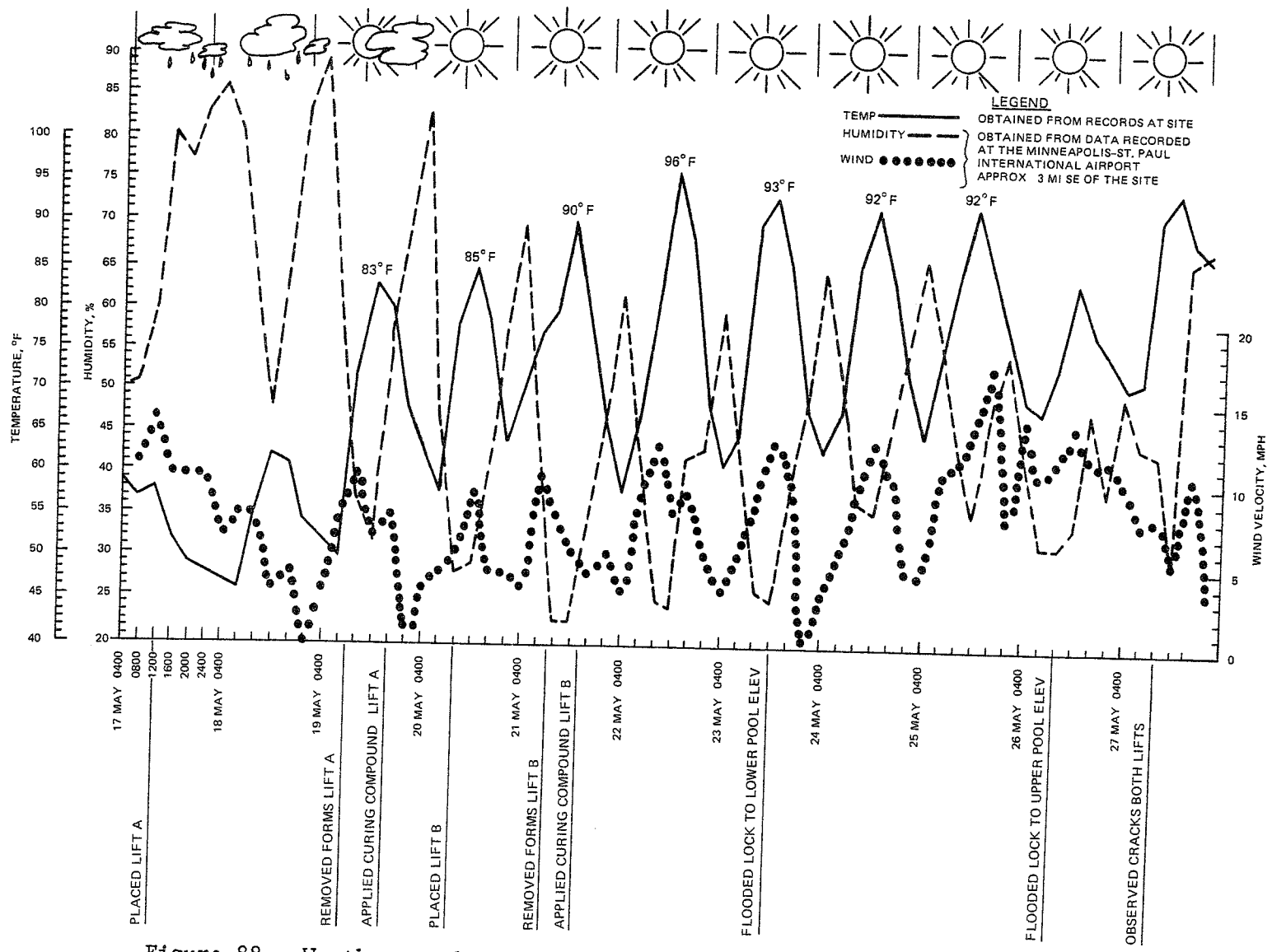


Figure 88. Weather conditions during curing of trial resurfacing monolith, Lock No. 1, Mississippi River

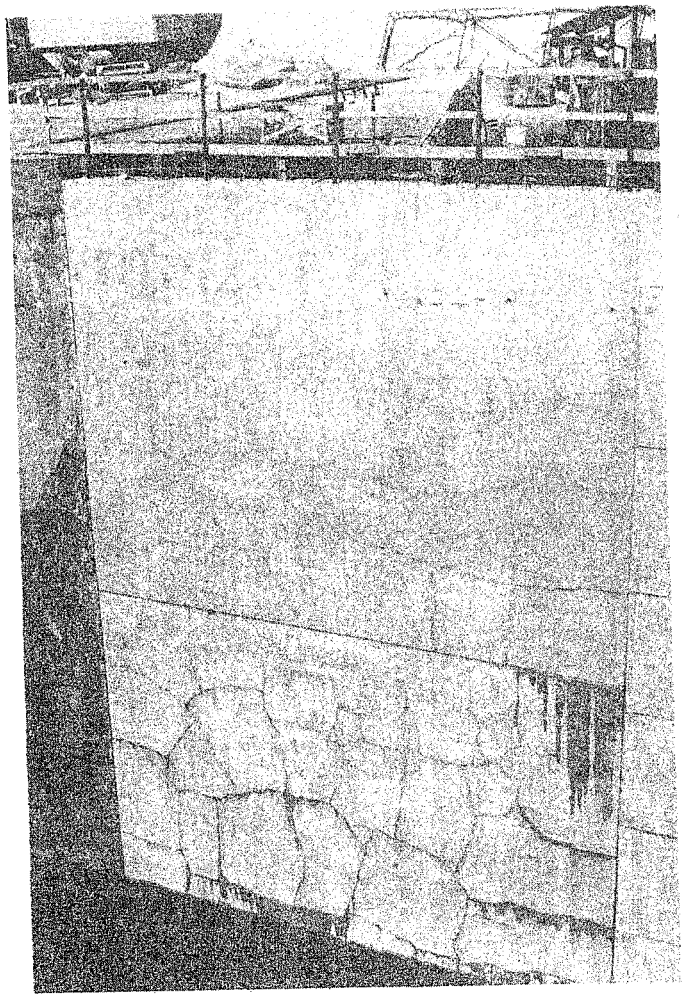


Figure 89. Cracking in replacement concrete, trial resurfacing monolith, 10 June 1980, Lock No. 1, Mississippi River

examination of the cores verified that good bond was obtained between old and new concrete. The examination also indicated that cracking initially developed at the face of the wall and propagated inward to approximately 90 percent of full depth of the resurfacing. There was no indication of reflective cracking although two vertical cracks at the top of lift B were in line with existing cracks in the old concrete. Results of compressive strength tests on selected cores and cylinders cast at the time of placement were as follows:

		Average Compressive Strength, psi	
Age, days	Specimen Type	Lift A	Lift B
7	Cylinder	4,660	4,690
28	Cylinder	6,100	5,180
59	Core	--	6,590
62	Core	7,110	--

207. It was specified that temperature reinforcement be located 4-in. from the finished lock face. Concrete cores showed the reinforcing steel to be located from 6 to 7 in. from the wall face. Measurements taken after placement of lift B showed the reinforcement at the top of the lift was located from 4-1/2 to 13 in. from the lock face (Figure 90). The improperly positioned reinforcement would have provided little resistance to shrinkage and temperature stresses.

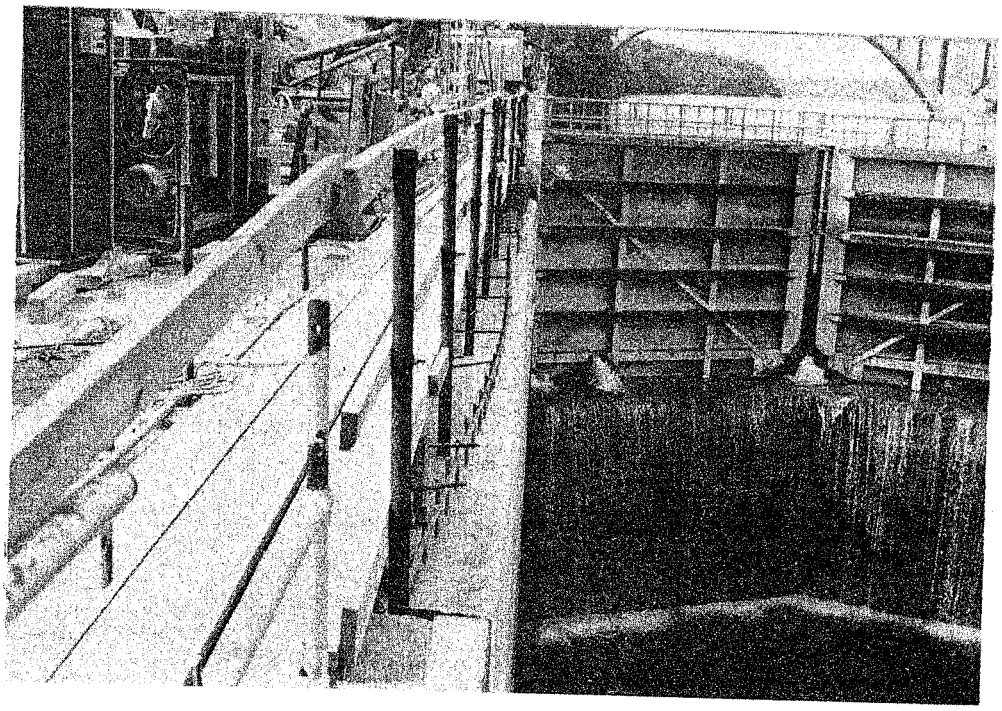


Figure 90. Location of reinforcing steel at top of Lift B, trial resurfacing monolith, 9 July 1980, Lock No. 1, Mississippi River

208. At 1915 hr on 22 May, a portion of the stoney valve house in monolith L-3 was demolished by explosives. It is not known what effect, if any, the blast may have had on the new resurfacing concrete in monolith L-5. The blast had no visible effect on recently placed concrete being used to reconstruct an adjacent monolith, L-4 (Figure 86). The most recent lift in monolith L-4 had been placed the afternoon of 21 May and consisted of 78 cu yd of concrete with a 5-ft lift height.

209. At the time of the trial placements, it was deemed unnecessary to install temperature probes for monitoring the concrete curing process. However, in previous placements where concrete temperatures were monitored, the 560-6PA6 pump mixture produced peak curing temperatures of 104 to 120°F or 40

to 50°F heat rise above placing temperature. Assuming equivalent heat was generated during the trial resurfacing, a peak temperature of approximately 115°F would have been attained. Also, based on the previous curing date, it was assumed that the concrete temperature in the trial lifts was still in the 85 to 95°F range on 26 May when the lock was flooded with water having a temperature of 50 to 50°F. Therefore, the thermal shock resulting from lock flooding may have been sufficient to cause or accelerate the concrete cracking.

210. While the effects of curing temperatures, adjacent blasting, and flooding of the lock were considered highly subjective, the fact remained that by any construction standard, the concrete was left unprotected from the sun, without any curing medium being applied, for an unacceptable period of time. Therefore, the District concluded that the quality control for the trial resurfacing was such that the exact cause of the cracking could not be ascertained. Also, that the project specifications appeared to be adequate to obtain concrete with a minimum of cracking. The following procedures, all within project specifications, were considered to be especially necessary in obtaining good results for the overlay concretes:

a. Use 1-1/2-in. maximum-size aggregate.

b. Use a concrete mixture proportioned for 3,000-psi compressive strength with a 0.48 water-cement ratio.

c. Reduce the amount of water-reducing agent to that recommended by the manufacturer.

d. Use proper and timely curing methods. If membrane curing is used, patching should wait until the curing period is completed.

e. The temperature of the fresh concrete should be as low as the project specifications allow.

f. Thoroughly consolidate placed concrete.

g. Use hot or cold weather placing techniques as appropriate.

211. During the second winter dewatering (1980-81), the new intake and discharge manifold systems were constructed, and the lock walls were resurfaced. Two upstream monoliths on both the intermediate wall and the upper guide wall were demolished and rebuilt (Figure 91) to accommodate the new lock intake system. Thus, the filling ports could be moved from the miter gate recesses, significantly reducing the amount of debris that collects in these recesses. Construction of the new discharge system involved five new mass

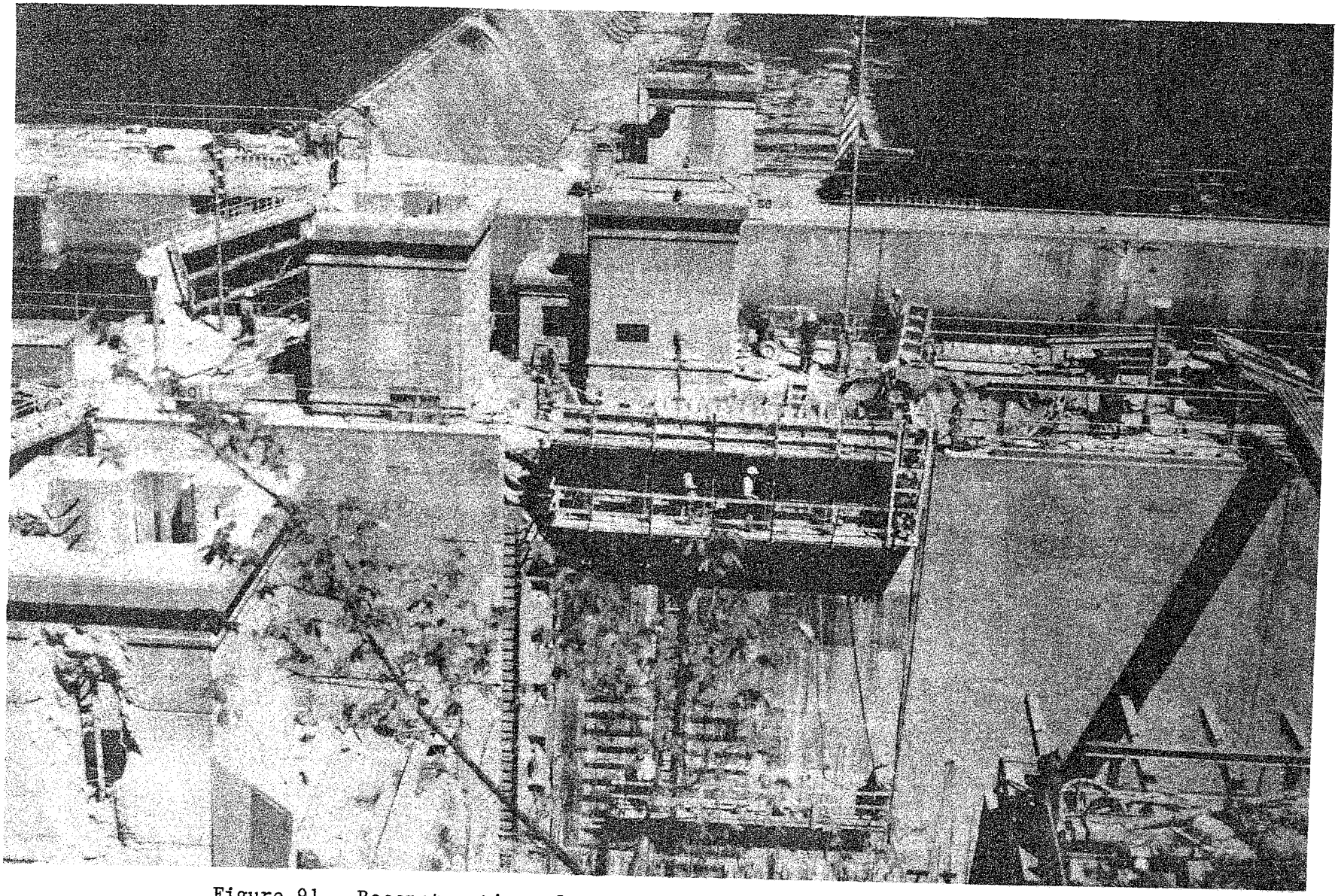


Figure 91. Reconstruction of monolith I-5, Lock No. 1, Mississippi River

concrete gravity monoliths for the intermediate wall, removal and reconstruction of four lower guide wall monoliths, plus placing a heavy reinforced concrete slab on the river bottom between the new intermediate wall monoliths and the reconstructed lower guide wall. Precast concrete units were then bolted to the slab to form the new lateral system. These units were connected to the wall ports on the new wall monoliths. Bulkhead slots were provided so that the laterals and the lock can easily be dewatered for future maintenance. Steel sheet-pile seepage cutoffs were driven around the slab to prevent erosion damage.

212. Plans for resurfacing of the vertical lock walls specified removal of 15 in. of concrete between elevations 732.7 and 686.2. A 3-in.-deep horizontal sawcut at elevation 686.2 was later changed by contract modification to a 6-in. depth in an attempt to prevent feathering at the bottom. During the winter of 1979-80, test blasts were conducted on one lock wall monolith to determine the proper explosive charge. Vertical holes, 2-1/2-in. diameter, were to be drilled along a line located 15 in. from the face of the lock wall. Holes were to be spaced 15 in. on center. The drill pattern was changed when the contractor requested permission to drill 18 in. from the lock face to allow himself additional tolerance in meeting the 13-in. minimum resurfacing criteria.

213. The initial test shot was conducted using four consecutive blast holes each loaded with 150-grain/ft detonating cord. Successive tests were conducted using four consecutive holes and increasing the explosive charge by 50 grain/ft until optimum results were obtained. The test results were of limited value in designing for full-scale removal because of the minimal number of loaded holes involved in each test round. However, it was determined that approximately 300-grain/ft detonating cord would be a reasonable charge to use in production removal. Blasting procedures for the 1980-81 wall resurfacing were designed by Woodward-Clyde Consultants. Their design incorporated the 300-grain/ft loading along with 400-grain/ft detonating cord attached to the bottom of the charges to make up for drill holes that may have wandered off line.

214. Results of the vertical wall removal (Figure 92) varied with respect to the condition and composition of the concrete being removed. Where excessive embedded form tie steel existed, large displaced concrete blocks or slabs frequently remained on the wall suspended by the form tie steel

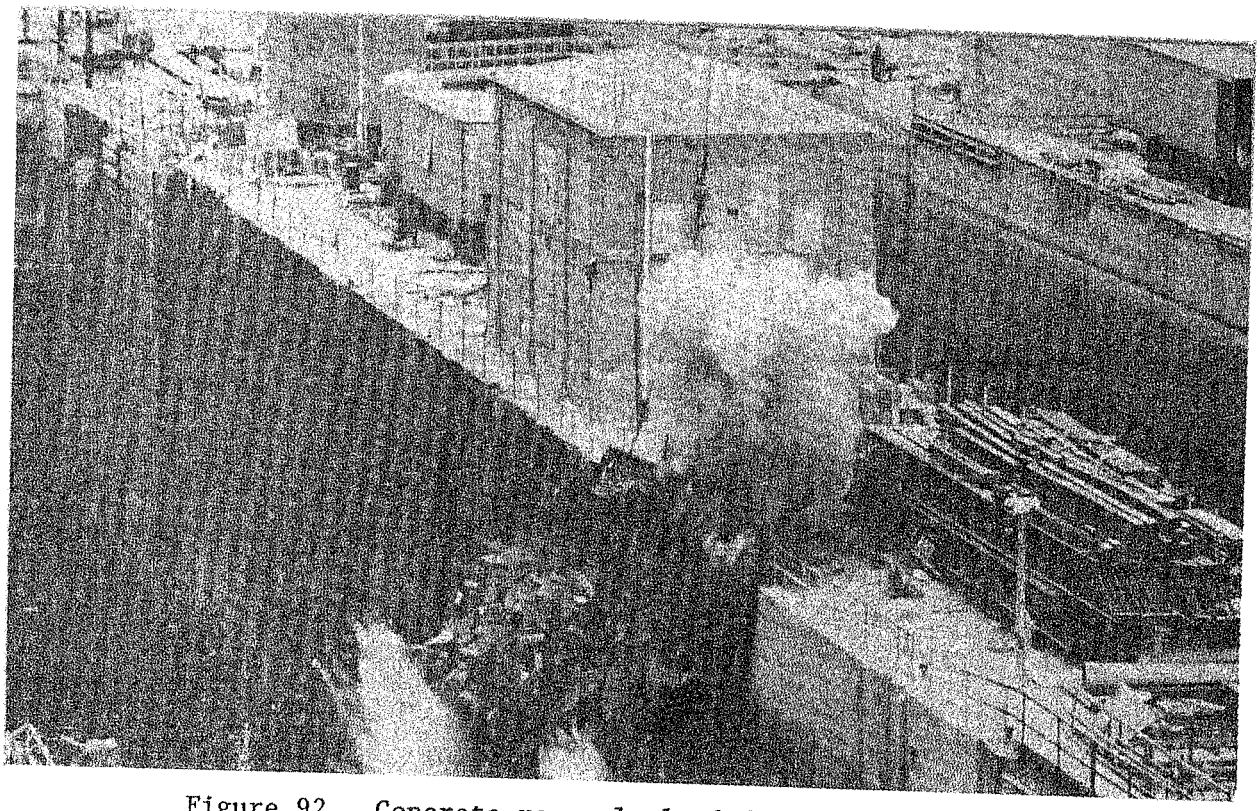


Figure 92. Concrete removal, land face of intermediate wall, Lock No. 1, Mississippi River

(Figure 93). These blocks occurred most frequently near the monolith joints. This hanging concrete presented a special removal problem requiring considerable removal effort by the contractor. Several "reshots" were necessary where these blocks remained, occasionally requiring even a third shot to obtain adequate removal. In an effort to alleviate this problem, the delays of some charges in the resurfacing shot were changed. However, this change came at a time when trouble causing form tie steel was no longer encountered in typical wall resurfacing blasting.

215. In most cases the concrete was removed at least to the level of the sawcut and in many instances feathered below that elevation. Some of the feathering below the sawcut was attributed to the patched and cracked concrete conditions left by the previous year's blasting of the lock filling and emptying ports. In some cases cracked and repaired areas came within inches of the wall resurfacing sawcut. Another probable cause of the feathering was that drill holes may have wandered excessively from their intended position or were drilled deeper than specified on the plans.

216. Prior to full-scale resurfacing, all aspects of the contractor's

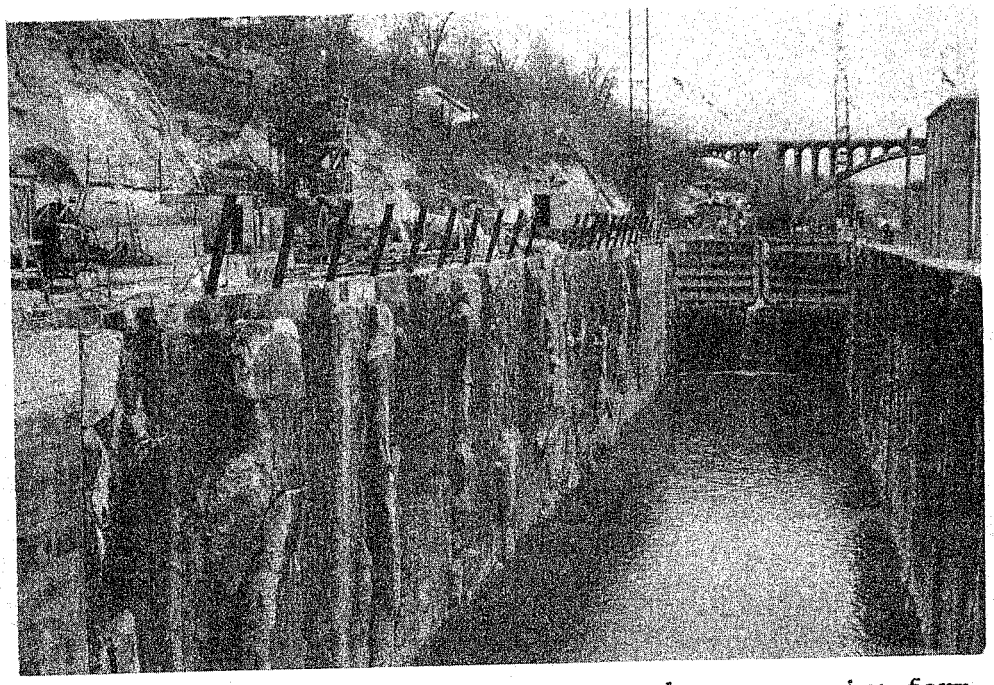


Figure 93. Hanging blocks of concrete where excessive form tie steel was encountered on the land wall, Lock No. 1, Mississippi River

performance were reviewed, and no major deficiencies were found. However, based on results of the trial resurfacing and Corps experiences with other lock wall resurfacing projects, it was anticipated that cracks would occur. In an effort to control such cracking, horizontal dummy joints 5 ft on centers were specified. These joints were specified to be a minimum of 3 in. deep (Figure 94) and to align with horizontal joints in the reconstructed tainter valve monoliths. Vertical joints were not specified because it was thought that they would be more vulnerable to barge impact. One of the resident engineers was assigned sole responsibility for monitoring the concrete placement and quality control to assure that good construction practices were followed within the limits of the specifications.

217. Following completion of concrete removal, a temporary truss and corrugated sheet metal structure provided by the Corps was placed over the lock chamber (Figure 95). This structure allowed the temperature within the lock chamber to be maintained at a minimum of 40°F during the winter at a site where snowfall is considerable and wind chill factors can range from -20 to -80°F. Hatches and construction elevators provided access from the ground surface to the floor of the lock chamber. The shelter was designed to be easily assembled and dissembled for reuse or salvage, making it an economical

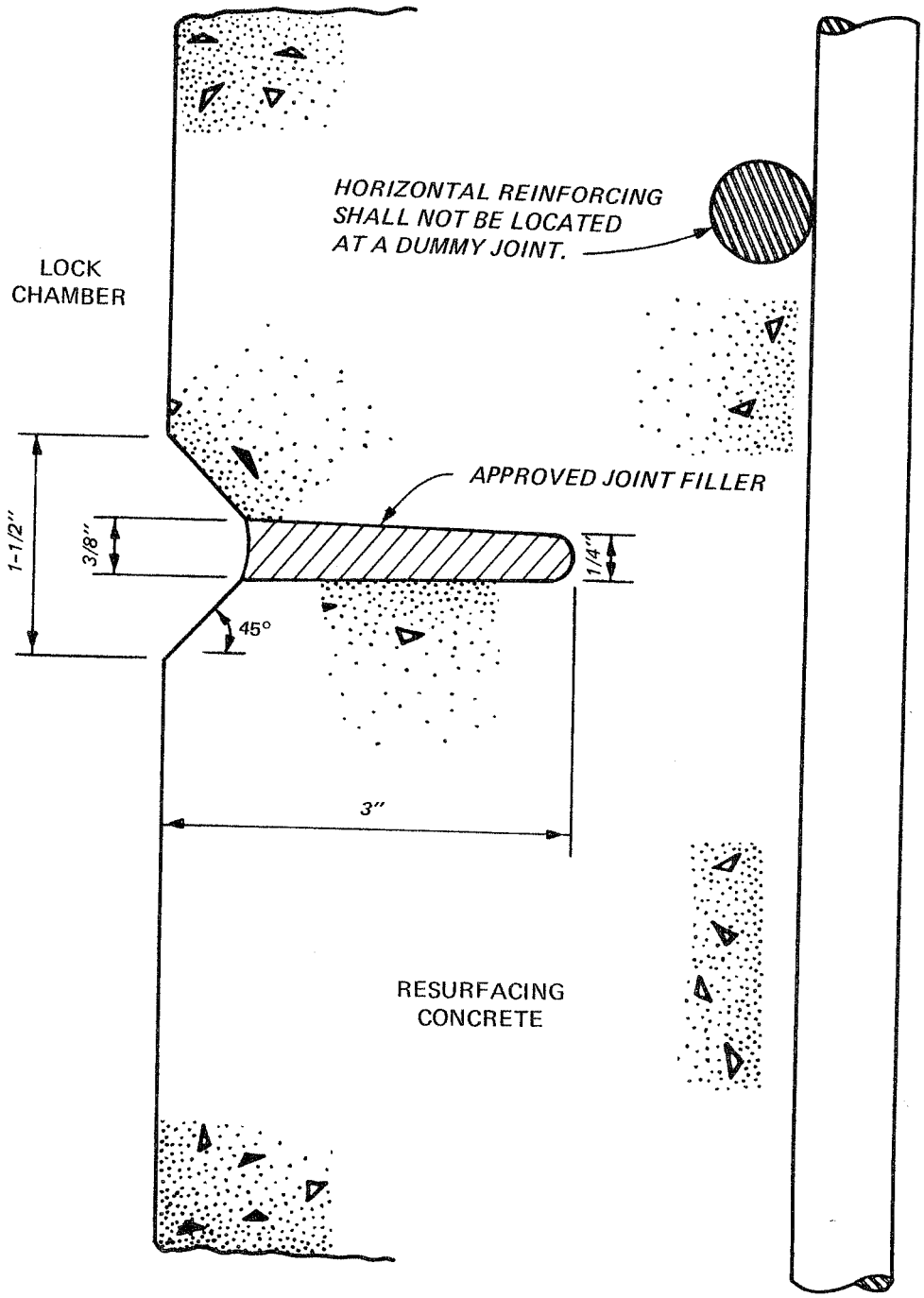
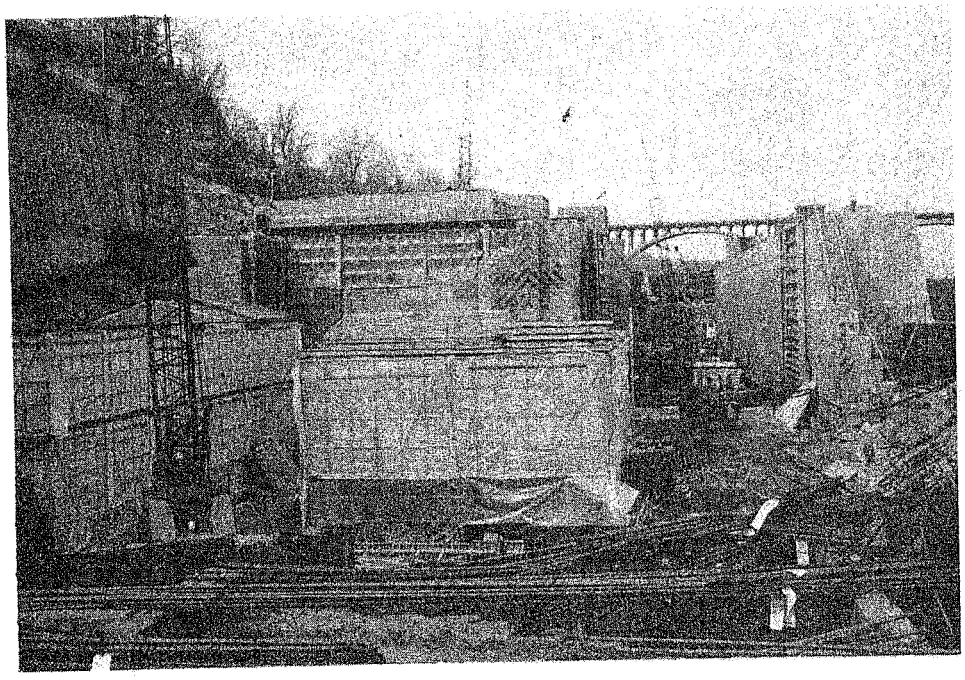
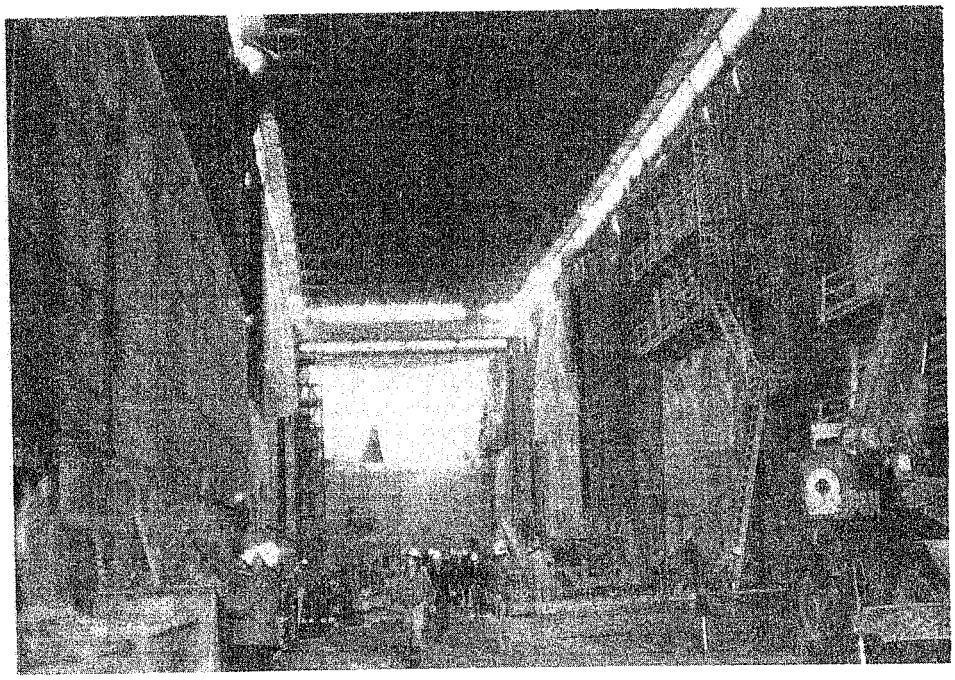


Figure 94. Horizontal dummy joint design, Lock No. 1, Mississippi River



a. Outside view



b. Inside view

Figure 95. Temporary enclosure over lock chamber, Lock No. 1, Mississippi River

solution for cold weather work. The shelter worked quite well during the investigative dewaterings, the test blasting program, and the Stage 1 contract during the first winter but had to be modified extensively the second winter to allow for wall resurfacing.

218. Surface preparation and replacement concrete reinforcement, anchors, and forms for the wall resurfacing were similar to that previously described for the trial monolith. Prior to placing the concrete forms, dummy joints were provided on the form face as shown in Figure 96. Waterstops and joint seals were installed at monolith joints as shown in Figure 97. Two lifts, each 23 ft high, were used to resurface each monolith.

219. A concrete mixture proportioned with natural gravel (1-1/2-in. MSA) for 4-in. slump, 5 percent air content, and a compressive strength of 3,000 psi at 28 days age was used in the wall resurfacing. Because of the slightly reactive river run aggregate, Type I low-alkali, portland cement was specified for the concrete. Materials were batched at the Shiely Concrete Company, St. Paul, Minnesota, and transported to the project in transit mixers. Concrete mixture proportions, based on a 1 cu yd batch, were as follows:

Mixture No. 510-4PA5

Material	Weight
Portland cement, type I	510 lb
Fine aggregate	1,275 lb
Coarse aggregate (1-1/2 - 3/4 in.)	1,010 lb
Coarse aggregate (3/4 - No. 4)	1,010 lb
Water	235 lb
Water-reducing admixture	19.6 oz
Air-entraining admixture	4.2 oz

Actual compressive strengths were significantly higher than the design strength with test results averaging approximately 3,800 and 5,040 psi at 7 and 28 days age, respectively.

220. Generally, the fine aggregate and water were preheated to approximately 130°F prior to introduction into the mixer. According to the producer, this step was necessary to avoid problems in discharging the concrete on cold days. As a result, concrete temperatures at placement averaged approximately 60°F. The specifications allowed concrete temperatures at placement to range

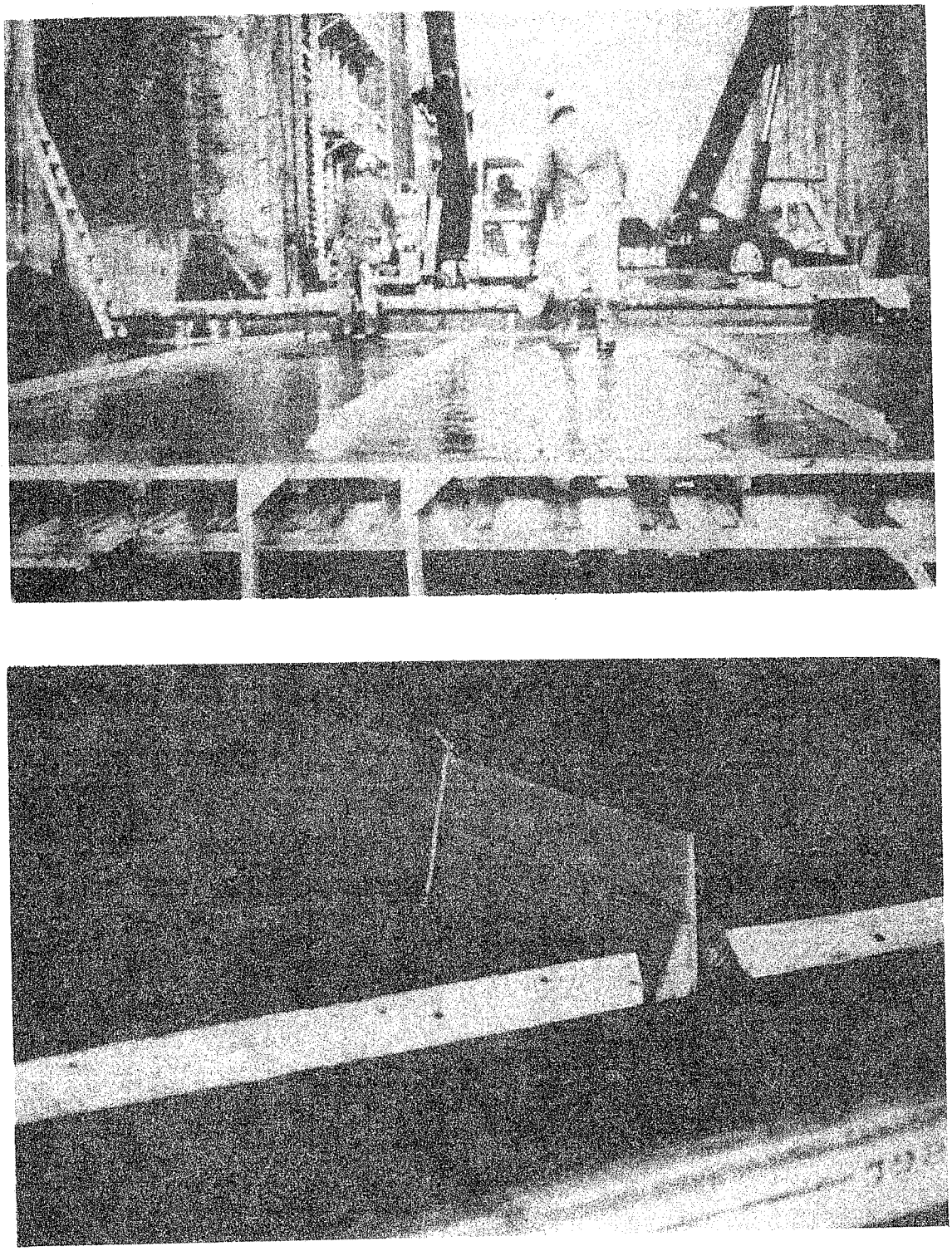
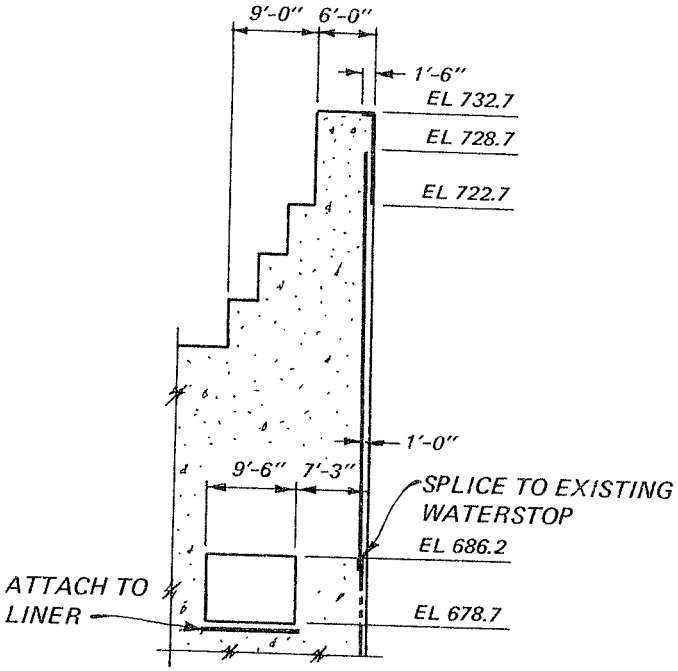
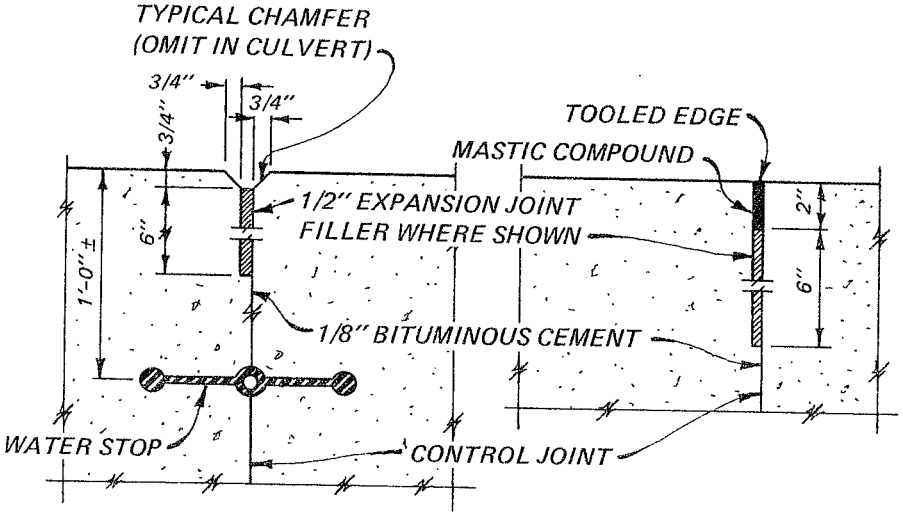


Figure 96. Forming for horizontal dummy joints, Lock No. 1, Mississippi River



a. Typical section



b. Typical joint detail except at top of wall

c. Typical joint detail at top of of wall

Figure 97. Waterstop and joint seal locations and details, Lock No. 1, Mississippi River

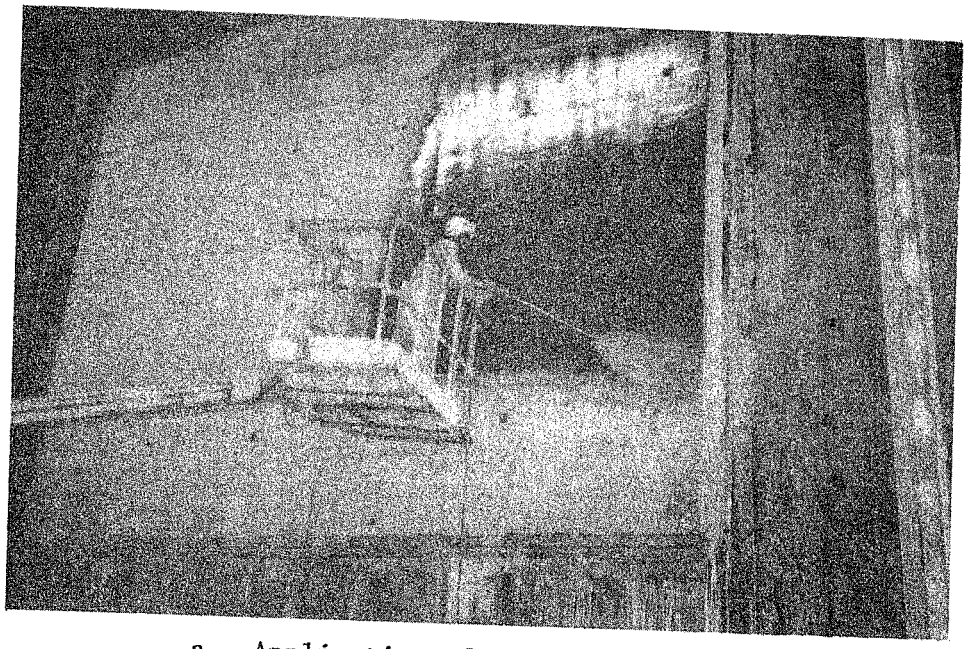
from 50 to 75°F. Ambient temperature inside the construction enclosure averaged approximately 45°F, or 5°F higher than the minimum specified.

221. After the forms were removed, generally at 1 to 3 days following concrete placement, the contractor had the option of using either moist-curing or a membrane curing compound conforming to CRD-C 300. Membrane curing was used predominantly because of the cold weather conditions. Following application of the curing compound, a polyethylene sheet was draped over the concrete surface for the duration of the curing period (Figure 98).

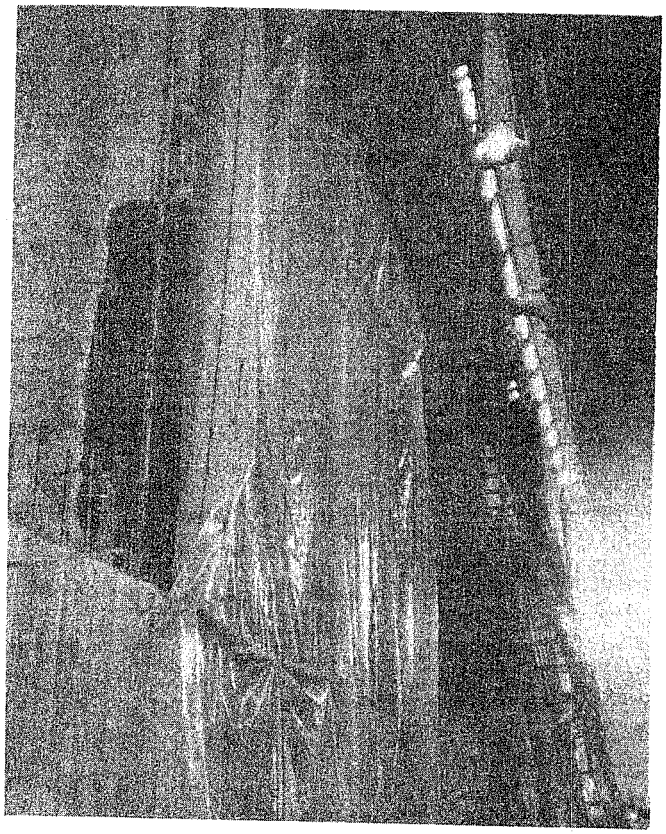
222. During the specified 7-day curing period, a temperature differential between the concrete surface and 2 in. inside the concrete of not more than 25°F was specified. The temperature probes and the recording meter were supplied by the Corps for contractor use. The probes were installed at locations determined by the Corps. Generally, temperatures were monitored by the contractor at 2-hr intervals throughout the required curing period or until such time that the Corps inspectors permitted a longer time interval. Typical results of the temperature monitoring on mixture 510-4PA5 are shown in Figures 99-102.

223. Results of the temperature monitoring of placement L-7B are shown in Figures 99 and 100. Although the form was not removed until 6 days after the concrete was placed, cracks were observed on top of the lift during the second day after placement. Temperature probes were located at two levels within the placement, 2 and 9 ft below the top of the lift. Peak concrete temperatures, approximately 24 hr after placement, were 95° and 98°F for the upper and lower levels, respectively, or 45° and 48°F higher than the average enclosure temperature. At the same time, the concrete surface temperature on top of the lift was reported to be 68°F. At the time cracks were observed, the temperature differential between ambient and concrete at 2-in. depth as measured from the wall face was 17° and 30°F for the upper and lower level probes, respectively. However, a temperature probe embedded 2 in. below the exposed top of the lift would have provided a more meaningful comparison in this case where the form was left in place for an extended period.

224. Results of temperature monitoring of placement L-11A are shown in Figure 101. A peak temperature of 106°F was recorded, 27 hr after placement, which represents a heat rise of 43°F. The form was removed at this point subjecting the concrete to a temperature differential between ambient and 2-in. depth of 56°F. Covering the concrete surface with polyethylene quickly



a. Application of curing compound



b. Concrete surface covered with polyethylene

Figure 98. Typical curing of concrete resurfacing, Lock No. 1, Mississippi River

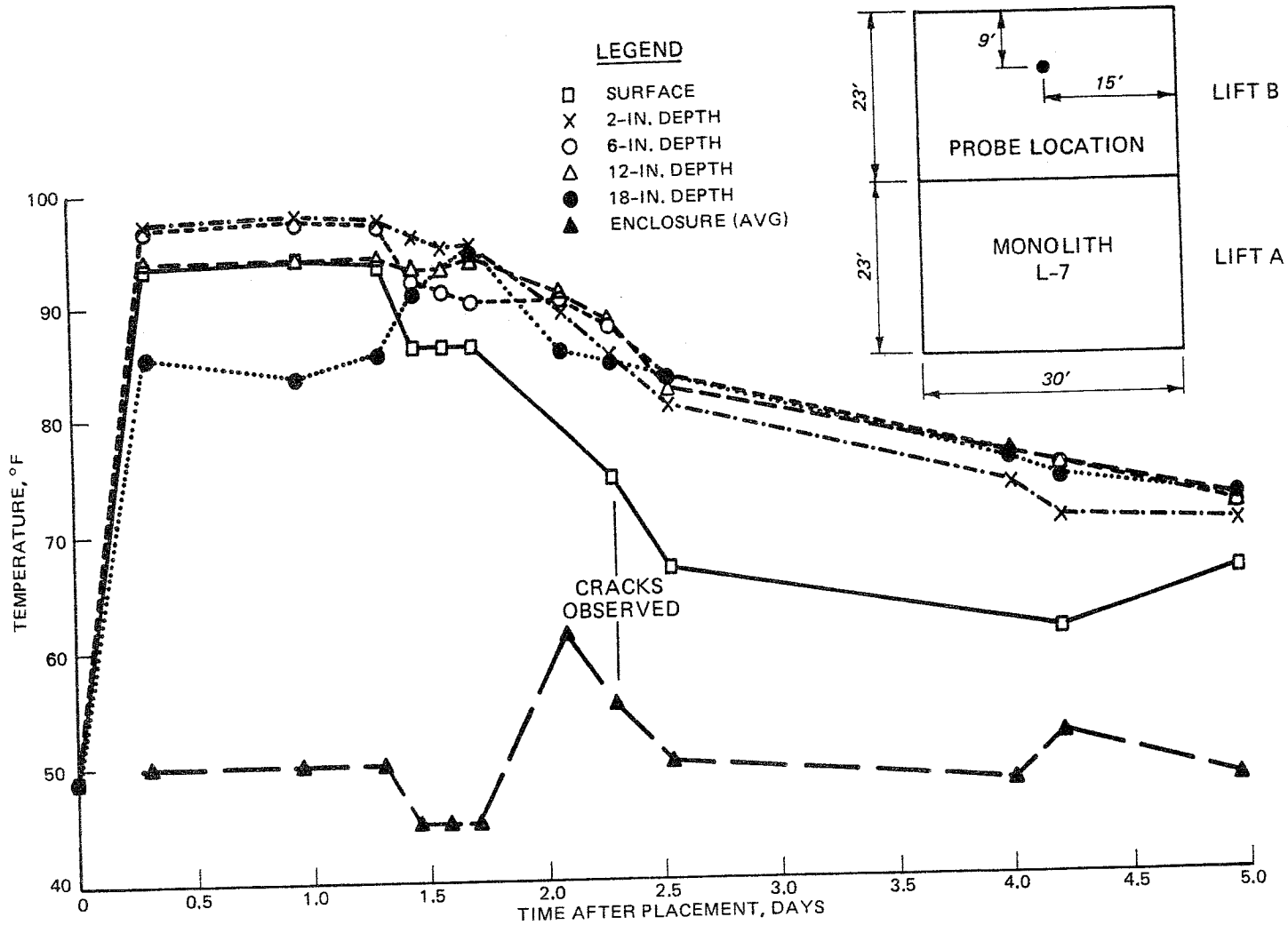


Figure 99. Results of temperature monitoring, mixture 510-4PA5, placement L-7B, 9 ft below top of lift B, Lock No. 1, Mississippi River

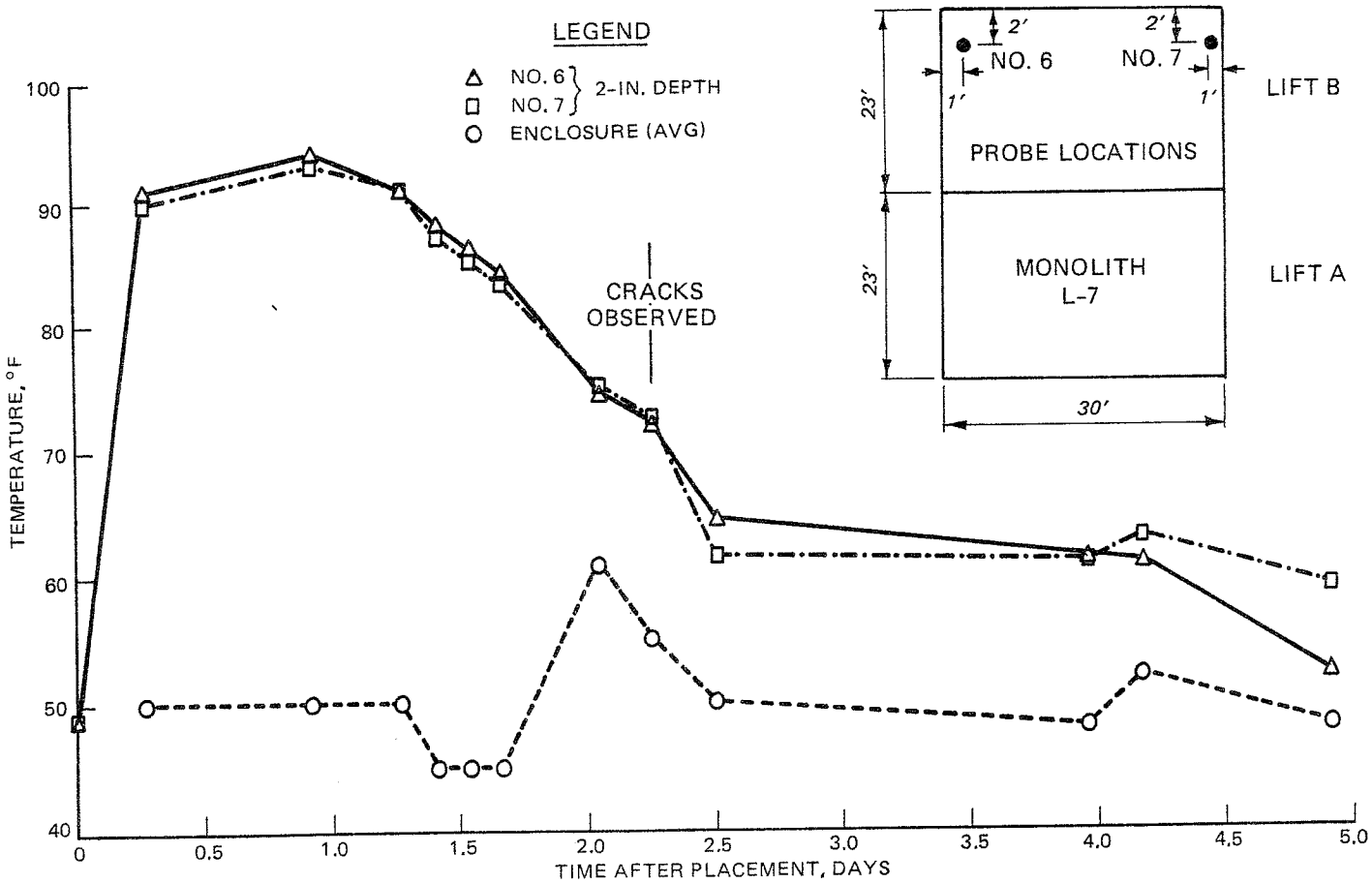


Figure 100. Results of temperature monitoring, mixture 510-4PA5, placement L-7B, 2 ft below top of lift B, Lock No. 1, Mississippi River

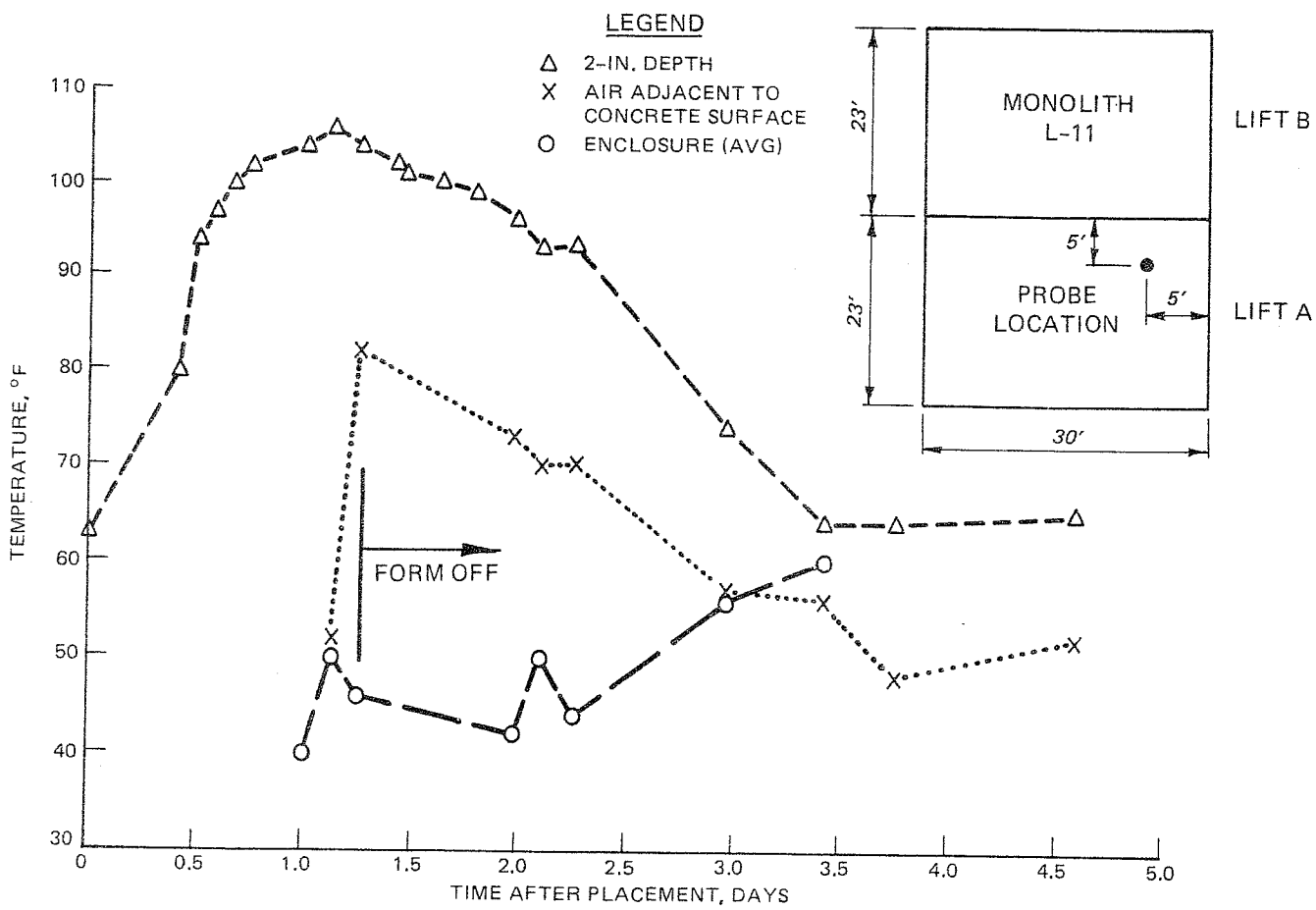


Figure 101. Results of temperature monitoring, mixture 510-4PA5, placement L-11A, Lock No. 1, Mississippi River

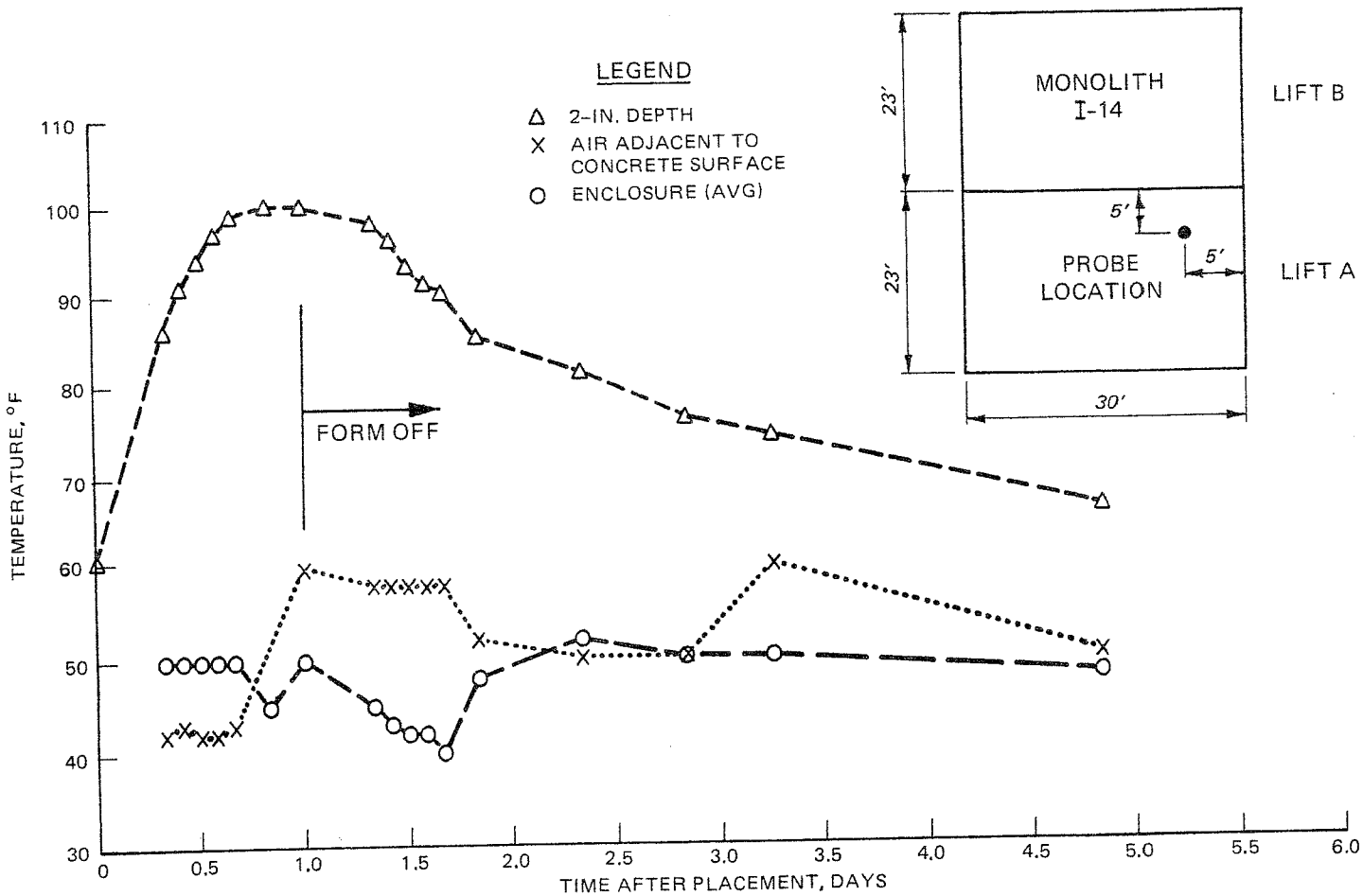


Figure 102. Results of temperature monitoring, mixture 510-4PA5, placement I-14A, Lock No. 1, Mississippi River

reduced the temperature differential between air adjacent to the surface and at 2-in. depth inside the concrete to 22°F. Similar results were obtained for placement I-14A (Figure 102). The form was removed one day following placement, subjecting the concrete to a temperature differential between ambient and 2-in. depth of 55°F. After the concrete surface was covered with polyethylene, the temperature differential between air adjacent to the surface and concrete at 2-in. depth was 40°F. This temperature differential decreased with time; however it remained in excess of 25°F for approximately two days. Although no cracks were reported for either the L-11A or I-14A placements, it is quite possible that they were present but not observed because of the polyethylene covering. Later reports that most placements contained cracks would appear to substantiate this possibility.

225. HQUSACE Division, District, and WES representatives met on 26 February 1981 to inspect the lockwall resurfacing operations. Approximately 35 percent of the resurfacing work was completed. Overall, the replacement concrete appeared to be satisfactory; however there were some hairline cracks evident. These cracks were generally vertical at about 5-ft intervals. The cracks were fine and the width in general was less than 0.001. Such cracks appeared to have no structural significance, and the inspection team agreed that they need not be repaired at the time. However, it was recommended that the cracks be mapped and monitored.

226. The dummy joints appeared to be controlling horizontal cracking. One disadvantage of the dummy joints is that their depth (3 in.) is such that the reinforcing mat located 5 to 6 in. from the lock wall face reduces its effectiveness in crack control. It would have been preferable to maintain a maximum of 4-in. cover, which could have been obtained by using a smaller depth to the dummy joints, or placing an additional mat of steel within the joints closer to the surface.

227. A number of approaches to minimize concrete cracking (such as reduced placement temperatures, use of fly ash as a partial cement replacement, metal forms instead of plywood, heating blankets, and vertical dummy joints) were discussed. However, the pressure to complete concrete operations prior to reopening of the lock scheduled for 1 May was such that there was a reluctance to make any significant changes in current procedures other than attempting to enforce the specified maximum temperature differential between air and concrete. There were plans to place the lower lift of one monolith using

Type K cement as an experiment. A suggestion that the experiment be expanded to include placing the remainder of the monolith with a concrete mixture containing 25 percent cement replacement material was taken under consideration.

228. On 17 March 1981, monolith L-7A was resurfaced with a concrete mixture containing shrinkage compensating cement (Type K). The mixture was proportioned with 1-1/2-in. MSA and a maximum water-cement ratio of 0.53 to have a compressive strength of 3,000 psi at 28 days age. Mixture proportions, based on a 1-cu yd batch, were as follows:

Mixture No. CC552-4PA5

Material	Weight
Type K cement	552 lb
Fine aggregate	1,162 lb
Coarse aggregate (1-1/2 - 3/4 in.)	868 lb
Coarse aggregate (3/4 in. - No. 4)	1,070 lb
Water	270 lb
Water-reducing admixture	22 oz
Air-entraining admixture	5 oz

229. A total of 47-1/2 cu yd of the concrete containing shrinkage compensating cement was placed. Average properties of the fresh concrete determined prior to pumping were as follows:

Property	Result
Slump, in.	4-3/8
Air content, %	7.3
Temperature, °F	47

Compressive strength test results averaged 3,730 and 5,030 psi at 7 and 28 days age, respectively.

230. Results of the temperature monitoring of placement L-7A are shown in Figures 103 and 104. The peak temperature recorded, 24 hr following placement, was 96°F, which represented a heat rise of 38°F. From removal two days after placement, the concrete was subjected to a temperature differential between ambient and 2-in. depth of 25°F. This differential decreased only slightly during the remainder of the curing period. Cracking similar to that previously observed in the conventional concrete resurfacing was reported. The cracks were not observed until the end of the curing period.

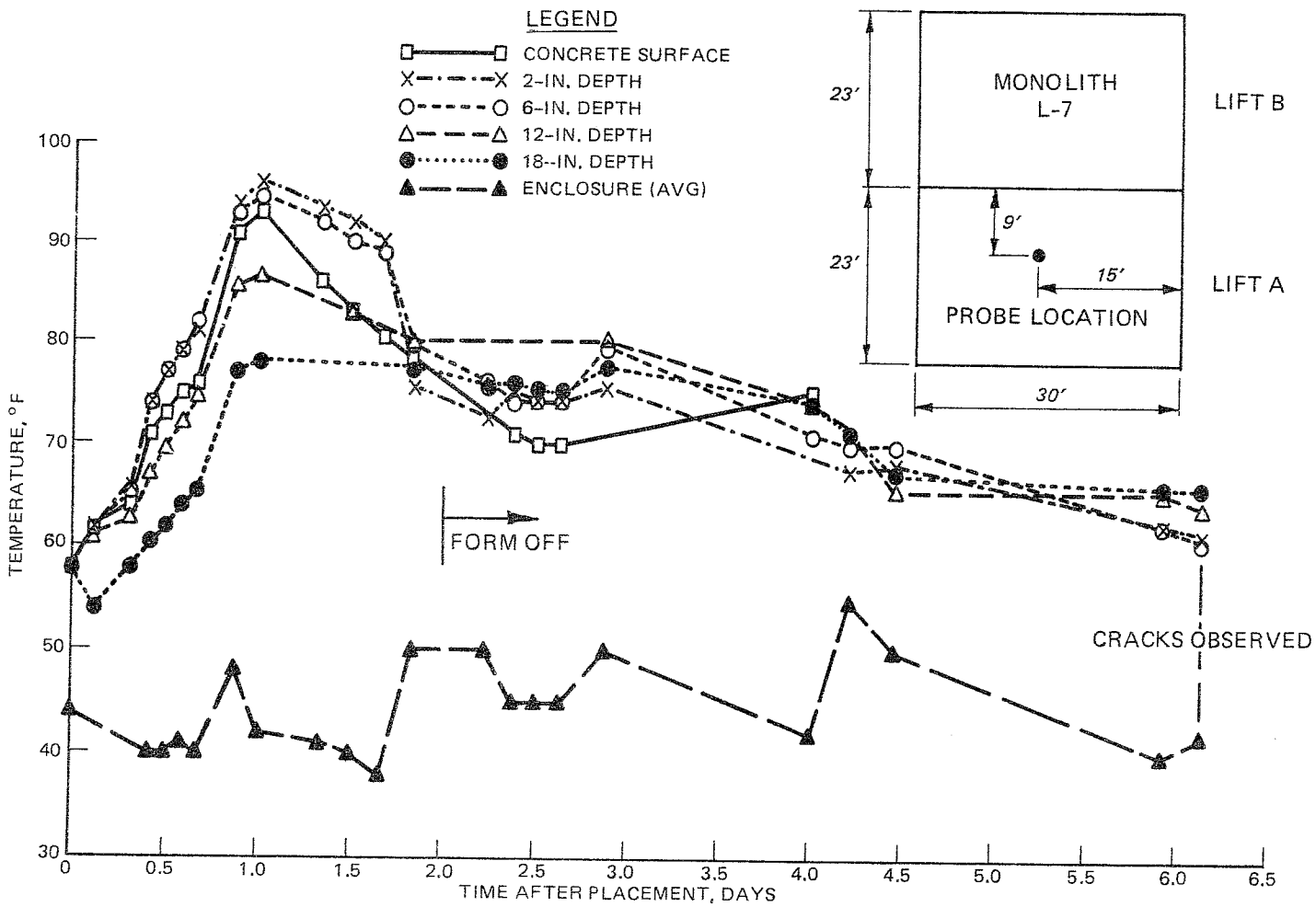


Figure 103. Results of temperature monitoring, mixture CC552-4PA5, placement L-7A, 9 ft below top of lift A, Lock No. 1, Mississippi River

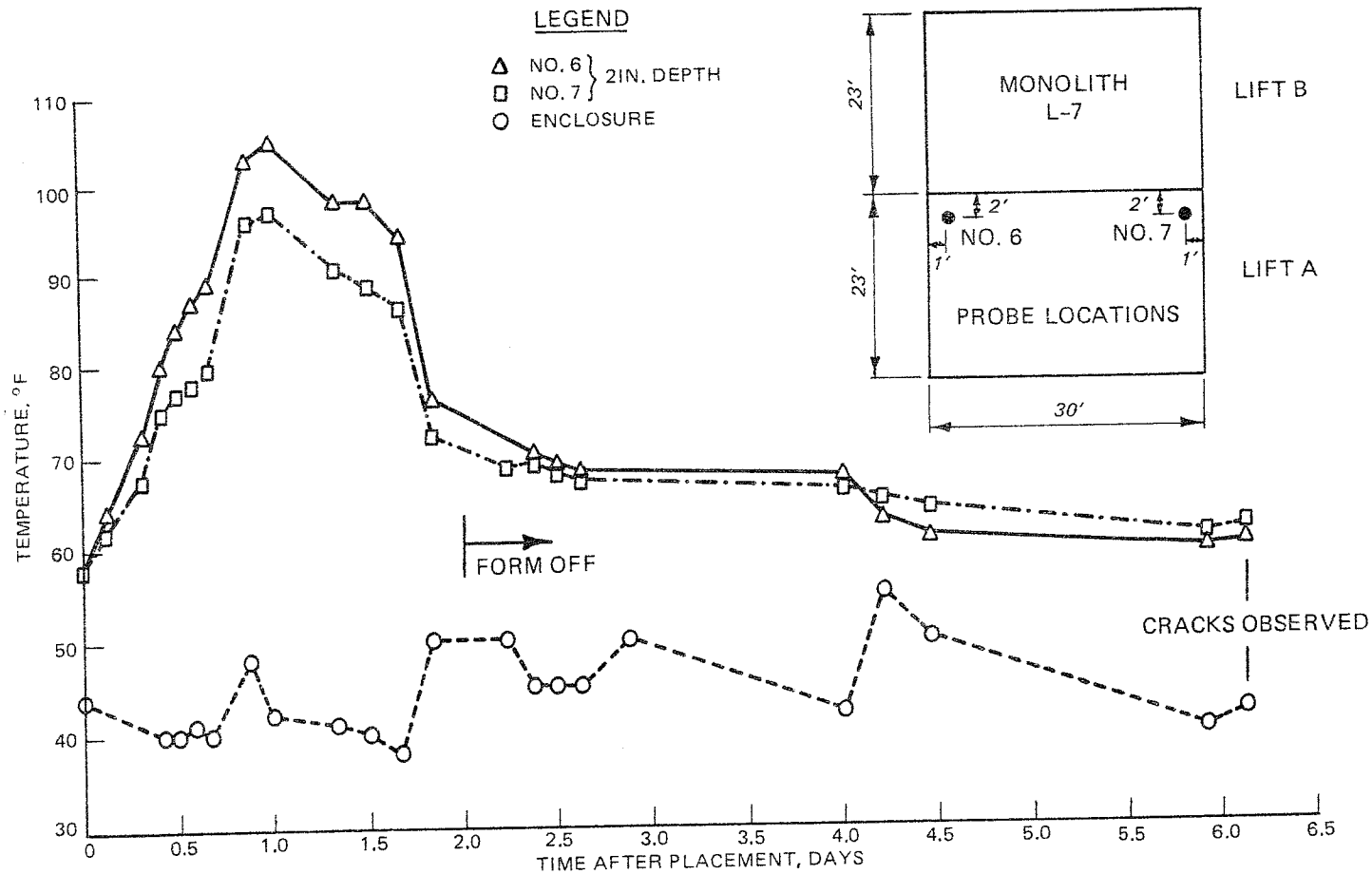


Figure 104. Results of temperature monitoring, mixture CC552-4PA5, placement L-7A, 2 ft below top of lift A, Lock No. 1, Mississippi River

231. On 26 March 1981, monolith L-10B was resurfaced with a concrete mixture containing fly ash. The mixture was proportioned with 1-1/2-in. MSA and a maximum water-cementitious ratio of 0.48 to have a compressive strength of 3,000 psi at 28 days age. Mixture proportions, based on a 1-cu yd batch, were as follows:

Mixture No. 78x517-4P5A5

Material	Weight
Portland cement, type I	439 lb
Fly ash	78 lb
Fine aggregate	1,070 lb
Coarse aggregate (1-1/2 - 3/4 in.)	1,060 lb
Coarse aggregate (3/4 in. - No. 4)	1,070 lb
Water	235 lb
Water-reducing admixture	25.8 oz
Air-entraining admixture	5.2 oz

232. A total of 44 cu yd of the concrete containing fly ash was placed. Average properties of the fresh concrete were as follows:

Property	Result
Slump, in.	2-1/2
Air content, %	4.3
Temperature, °F	55

Compressive strength test results averaged 4,280 and 5,150 psi at 7 and 28 days age, respectively.

233. Results of the temperature monitoring of placement L-10B are shown in Figure 105. The peak temperature recorded, 27 hr following placement, was 96°F which represented a heat rise of 41°F. At the time of form removal, approximately 1-1/2 days after placement, the concrete was subjected to a temperature differential between ambient and 2-in. depth of approximately 40°F. This differential exceeded 25°F for two days following form removal. Although cracks were not observed until four days after placement, it was noted that they may have occurred earlier.

234. Although horizontal cracking was controlled through the use of dummy joints, efforts to eliminate vertical cracks in the resurfacing were unsuccessful (Figure 106). The tainter valve monoliths, which were demolished

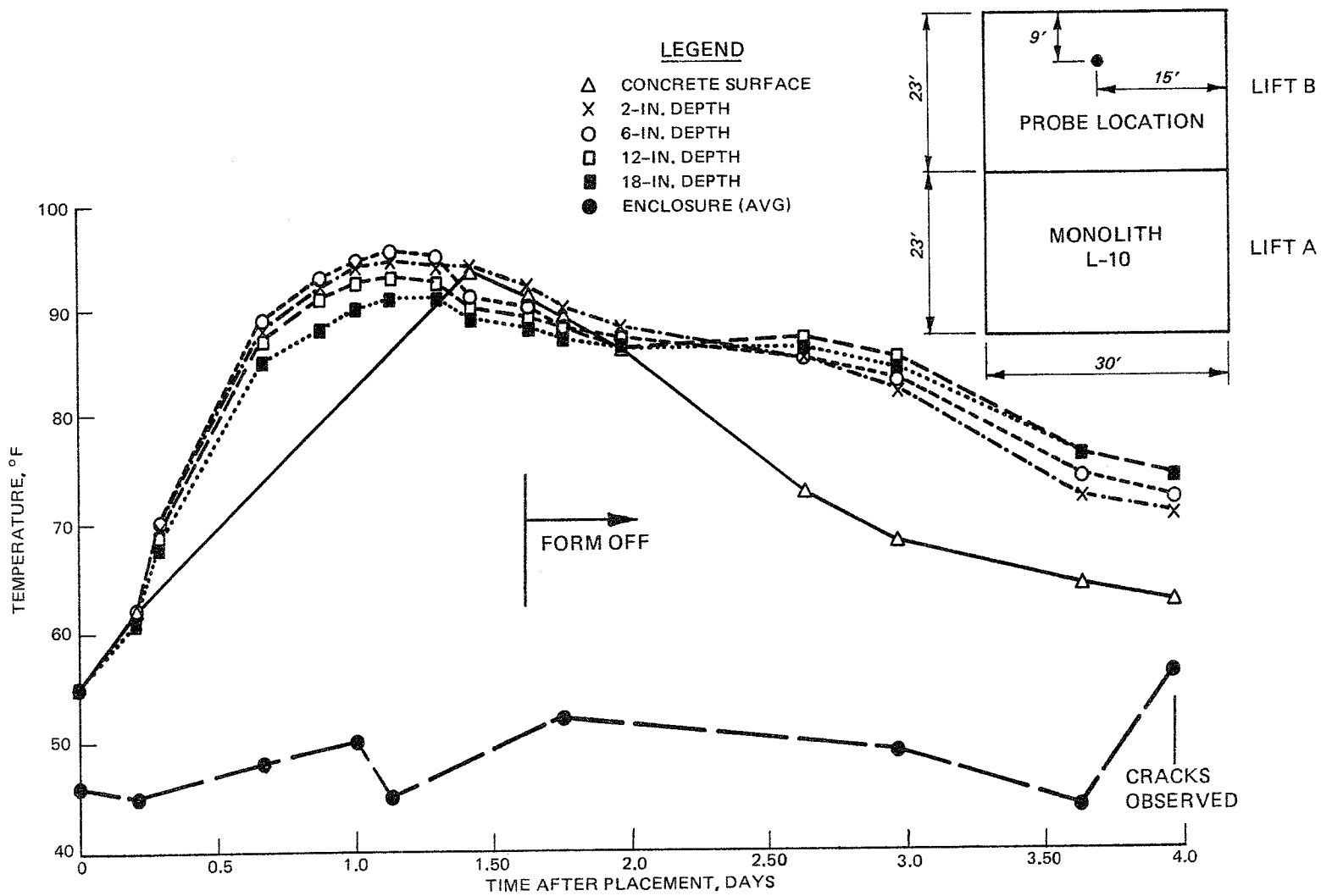


Figure 105. Results of temperature monitoring, mixture 78x517-4P5A5, placement L-10B, Lock No. 1, Mississippi River

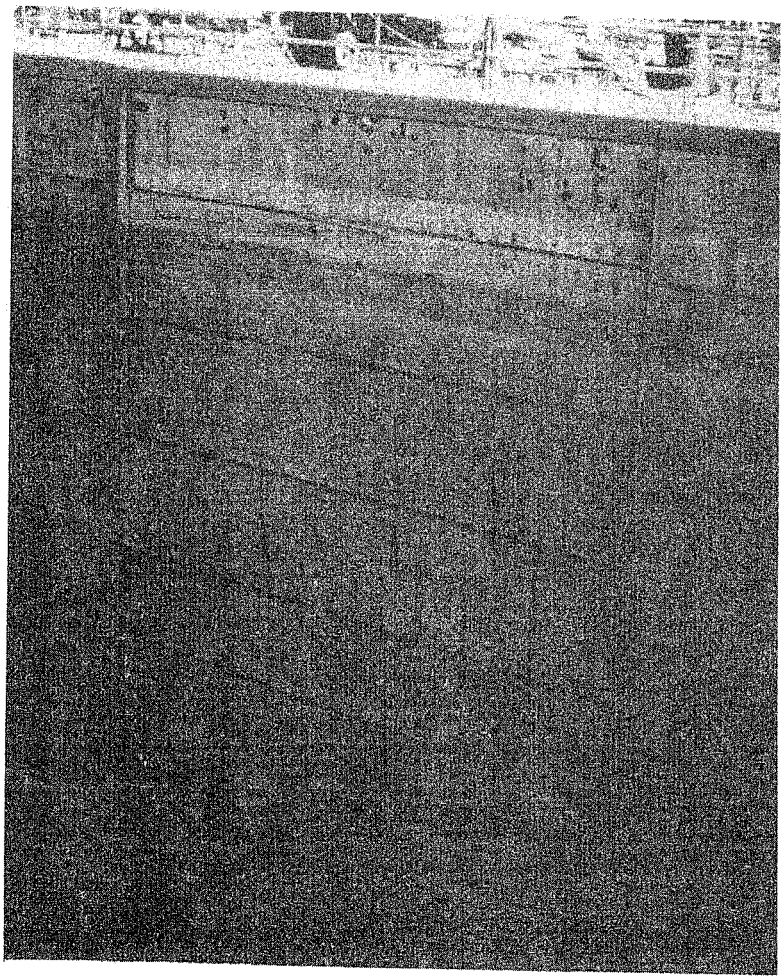


Figure 106. Vertical cracking in the concrete resurfacing, Lock No. 1, Mississippi River

and reconstructed in 5-ft lifts with essentially the same concrete materials and mixture proportions, exhibited a minimum of cracks (Figure 107). This fact suggests that restraint provided by the existing concrete was the primary cause of cracking. As the relatively thin layer of resurfacing concrete attempted to contract as a result of temperature, shrinkage, and autogenous volume changes, this restraint caused strains to develop in the concrete resurfacing which exceeded its tensile strain capacity. Consequently, cracks developed.

235. During both winter dewatering periods, approximately 150 people worked two 10-hr shifts 6 days per week from December through May. A total of 22,000 cu yd of concrete was placed utilizing 10 different concrete mixture designs. Three simultaneous concreting operations, each with different

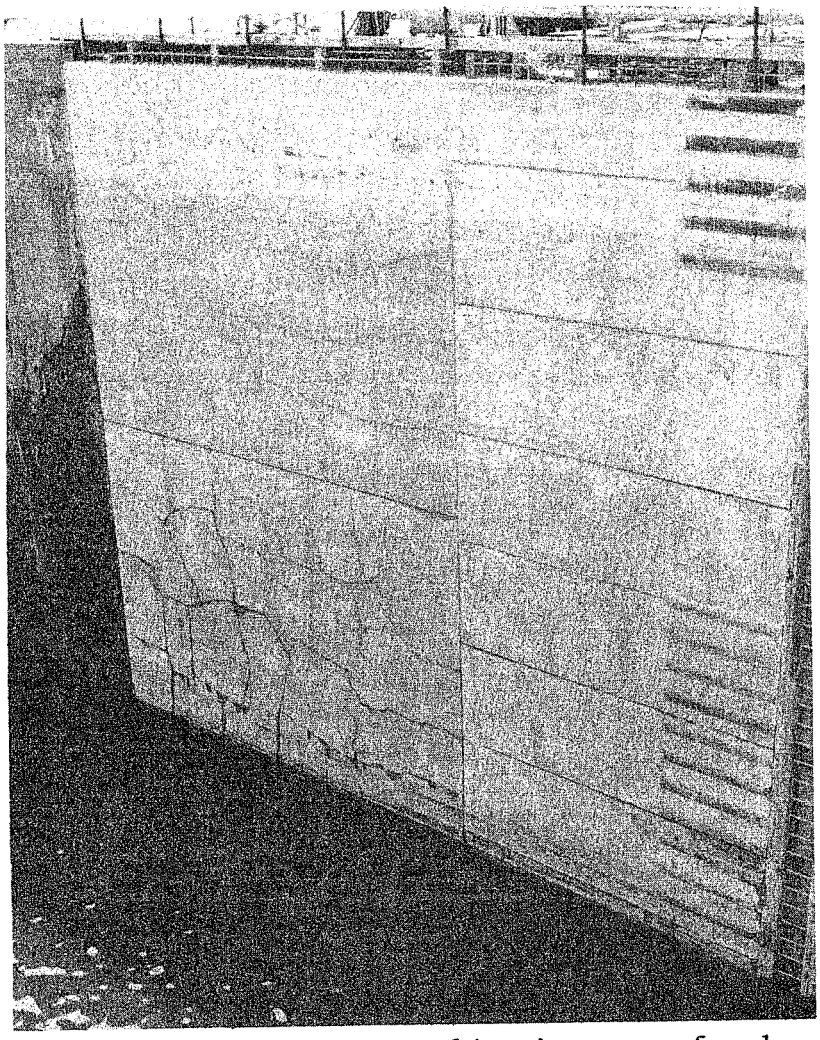


Figure 107. Concrete cracking in a resurfaced monolith (left) compared to reconstructed monolith, Lock No. 1, Mississippi River

placement methods and mixture designs, were commonplace. Through the diligent efforts of both the Corps and the Stage 1 construction contractor, the lock was closed to navigation for only the two 5-month winter dewatering periods, a total of 3 months beyond normal winter shutdowns. Impact of this closure time was lessened by close coordination with area shippers and carriers.

236. The second major construction contract (Stage 2) was awarded to M. A. Mortenson Company, Minneapolis, Minnesota, in October 1981. This contract included construction of a new central control station, service building, utility building, and associated electrical and mechanical systems. For visual integration with the new rehabilitated locks, the buildings were constructed of cast-in-place concrete faced with a cementitious coating.

237. The focal point is the central control station on the intermediate wall. Reached by a pedestrian bridge that also carries utilities, the building was designed to conserve energy. North facing windows are minimal, while south windows admit sunlight to the entire floor area during the winter, where water tubes absorb the solar gain. The windows are protected with insulated coverings at night. In summer, structural shading prevents sunlight from entering. An active solar system, which uses freon as a phase change material for the heat transfer medium, supplements the hot water system.

238. The central control station houses the computer system which controls all of the lock operations. The lock master is no longer required to go down to the lock wall in order to raise a filling valve or open a gate. Instead, an entire lockage can be completed from the control room overlooking the lock chamber. Those areas which cannot be seen directly from the control room are monitored by five remote television cameras.

239. The cost of the rehabilitation project increased from an estimate of $21,900,000 in the Study of Alternatives Design Memorandum prepared in October 1976 to approximately $45,000,000 in 1982. The District spent considerable time reviewing and analyzing the situations and reasons for this increase and concluded that the increase could be divided into eight categories (St. Paul District 1983). There is some overlap between categories, but the titles generally represent some of the problems and uncertainties in preparing an accurate cost estimate for a difficult project at the feasibility level.

a. Design revisions. Revisions to the feasibility design accounted for approximately 23 percent of the cost growth. As the detailed design was refined during the design memorandum stage and during the preparation of plans and specifications, revision were required. The more detailed design also allowed more accurate estimating techniques to be used. Errors and omissions in the calculations of early quantities were also corrected.

b. Inflation. A major contributor to cost growth was inflation. It amounted to approximately 16 percent of the total.

c. Safety improvements. The third largest growth factor of approximately 15 percent was for safety improvements. As the design proceeded and onsite investigations were made, several additional features were added to improve the safety of the project. Additional stability anchors were incorporated. The original design of the cellular sheet-pile cofferdam was modified. Hydraulic improvements were added as a result of the

hydraulic model study. Numerous smaller safety revisions were also included as the project design progressed.

d. Lock rehabilitation inexperience. Approximately 13 percent of the increase is attributable to the District's inexperience in engineering a major lock and dam rehabilitation. Because of this lack of experience, the early design was inadequate in some aspects. For example, cost associated with electrical features of the rehabilitation increased significantly from the original estimate. The extent of work required to convert electric equipment over a half century old to a modern, sophisticated computer-based system was simply not adequately accounted for. Other examples include a cost increase to properly account for the constricted time requirements and confined site conditions and the necessity to revise the size and scope of such features as the service building and sewer system.

e. Concrete removal by blasting. Large amounts of concrete had to be removed for lock rehabilitation, and the cost for this removal included in the original estimate was based on previous experience. However, as the detailed plans for lock rehabilitation were formulated and the emphasis was placed to minimum closure to navigation, it became apparent that the previous methods of concrete removal were not applicable. Approximately 12 percent of the cost growth is a result of additional cost for use of state-of-the-art blasting techniques for concrete removal. These costs include the need for research and development, an additional dewatering needed to test the blasting techniques, and as a modification to the construction contract to field-adapt the plans and specifications to site conditions revealed during construction. Additional expenses were incurred in the use of Title II consulting services for onsite inspection of the blasting techniques.

f. Fixed time schedule. The Minneapolis Port Authority and news media cited $20,000,000 in lost revenue for each month Lock and Dam No. 1 was closed to navigation. Congressmen wrote the District to encourage progress towards opening the lock. The District made a firm commitment for opening the lock at the end of each of the dewaterings and would have received severe criticisms if the dates had not been met, regardless of the problems. For these reasons, an additional 10 percent of the original estimated cost was spent to ensure the locks were open on time. In addition to two modifications for acceleration, the District specified some expensive, but expedient, construction methods.

g. Receipt of bids. Approximately 9 percent of the cost growth resulted from the bid opening of the Stage I contract and the operating machinery supply contract. The fall of 1979 was a good period for the construction industry and competition for work was not intense; therefore, contractors' bids were consistently higher than during periods of minimum construction

activity. Only two bids were received on the Stage I contract of this difficult project, and following award the project cost was revised upward.

h. General rehabilitation. It is difficult to determine how much should be set aside for contingencies on a rehabilitation project when so much depends upon the unforeseen. At Lock and Dam No. 1, approximately 2 percent of the costs were incurred beyond the assumed contingencies because of differing site conditions.

240. In spite of these problems, the rehabilitation of Lock and Dam 1 was successful. The project has won numerous engineering design awards, including the OCE Engineering Award of Excellence and the Presidential Design Award of Merit. The overall design and construction cost of $45 million represents only about one-fourth the cost of a new lock and dam structure. Yet the rehabilitated facility is as modern and efficient as any new lock.

Lower Monumental Lock

241. Lower Monumental Lock and Dam is located on the Snake River about 40 miles from Pasco, Washington. Initial construction began in 1961, and the project was put into operation in 1970. The navigation lock chamber has clear dimensions of 86 by 675 ft and a lift height of 103 ft (Figure 108).

242. The environment at Lower Monumental is harsh from the standpoint of concrete durability. The region does not have extreme winters during which the temperature drops below freezing and remains there. Instead, the concrete is exposed to many alternate cycles of freezing and thawing. This exposure is exaggerated when the water is just above freezing and the air temperature is below freezing. Freezing and thawing based on daily ambient temperature changes (a conservative estimate of total cycles of freezing and thawing) averages 64 cycles per year. In addition, lock usage averages about 475 commercial lockages each year during the months of freezing weather.

243. Deterioration and repair of the concrete in the lock wall are described in detail by Schrader (1981) and summarized in the following discussion. Aggregates for the concrete were natural materials screened from river deposits. During construction, it was necessary to use an unusually high dosage of air-entraining admixture in order to stay above the acceptable lower limit of required air content. The air content was checked at the time of batching and before placement. However, analysis of hardened concrete from

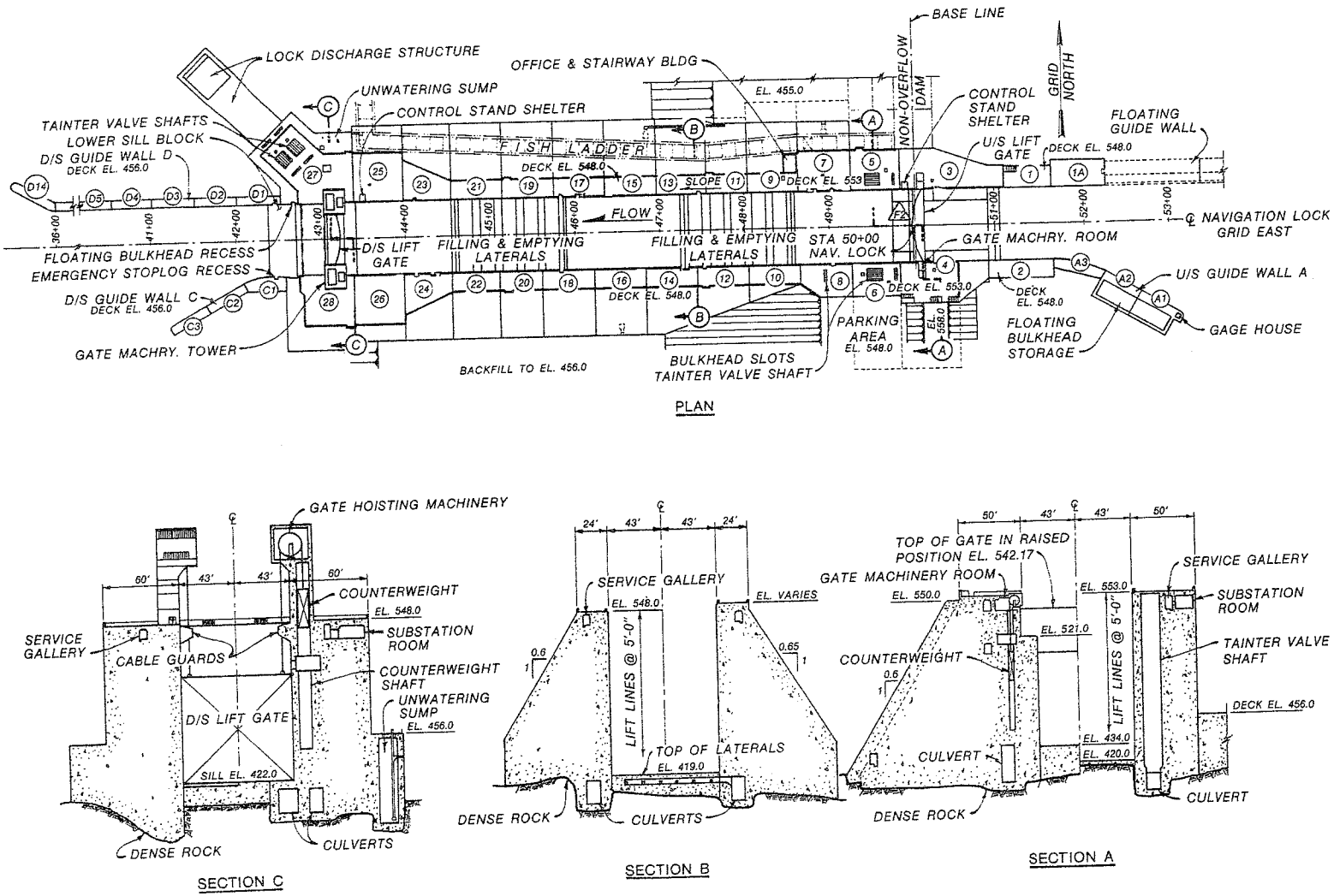


Figure 108. Plan and typical sections, Lower Monumental Lock

the lock indicated that the actual air content was less than required and that the concrete had very low resistance to cycles of freezing and thawing.

244. Concrete deterioration in the lock chamber was evident after several years of operation. It progressed through the next few years to the point at which it became obvious that repairs would be necessary. The lower areas of the lock chamber had the worst damage with 3- to 6-in. aggregate fully exposed after about eight years of service. Because of the very high lift, loose aggregate falling into the lock chamber created a potential safety problem. In addition to repairing the badly damaged areas, a treatment was needed that would prevent further deterioration of those areas not yet damaged to the point of needing repair.

245. The conventional approach to lock wall repair, removal of about 12 in. of face concrete and replacement with anchored high-quality concrete or shotcrete, was considered to be too costly and time-consuming. The lock, a major link on the Columbia and Snake River Waterway, has no alternate barge handling facility. Therefore, a closure for more than the routine two- or three-week annual maintenance outage could not be tolerated. A faster and more economical method of repair and prevention of further deterioration of the lock walls was needed. If, after proper surface preparation, a protective shotcrete coating could be applied to the existing wall, large savings in time and material would result.

246. Any buildout would have to be minimal so that the effective lock width would not be reduced. Areas of the structure where severe deterioration and spalling occurred to depths ranging from several inches to several feet would first be filled with concrete, epoxy mortar, shotcrete, or other patching materials. The surface coating would then be applied over the patch. In order to be successful, the coating would have to cure rapidly, bond to the existing wall, be resistant to wetting and drying and cycles of freezing and thawing, have dimensional stability and minimal shrinkage from moisture and temperature changes, prevent the penetration of water through it even at heads of 100 ft while still being able to breathe or relieve vapor pressure when the lock is empty, be resistant to the impact of barges, have acceptable appearance, be able to be applied at temperatures between 30° and 90°F on a surface that would be near a saturated surface dry condition, and be practical enough to apply in the field at a production rate of about 10,000 sq ft per day. A material meeting these prerequisites was not located.

247. The most promising idea was to apply a shotcrete that would contain modifiers which could enhance its qualities. The addition of fibers could add toughness, resilience, impact resistance, strain capacity, and other desirable properties. Impermeability, bond, rapid curing, and minimal shrinkage could be provided by a latex modifier. Portions of concrete from the lock were removed, coated with a 3/8-in.-thick fiberglass fiber-reinforced latex-modified mortar, and tested to see if this type of coating did, in fact, have potential for repair of the entire lock. Laboratory tests demonstrated that the specialized shotcrete coating had the potential to meet the criteria for the repair material.

248. The next steps in evaluation of the specialized shotcrete method of repair and protection were to demonstrate its field practicality, thoroughly test the material properties of field-cast panels, and observe the field performance after a year's exposure to actual operational conditions. Monolith 9 was selected for the field trials because the lock chamber face had various degrees of surface deterioration ranging from minimal at the top to severe at the bottom. It was divided into six equal 10-ft-wide strips running from the top of the lock down to tailwater. Each strip was coated with one of the trial mixture proportions. The field test was performed by a contractor under fixed bid in 1979.

249. The first, and perhaps most important step, was preparation of the existing surface. Contract specifications, as stated below, were clear as to what was to be accomplished during the surface preparation phase:

> "Prior to applying any of the shotcrete coatings, the surface shall be prepared by removing all loose, unsound, and friable material and by removing all surface contaminants such as dust, silt, old curing compound, organic growth, etc. The purpose of applying the coatings is to prevent continued deterioration of the mortar portion of the concrete. Due to deterioration that has occurred to date, much of the mortar is very poor, crumbly, and friable. All of this unsound material shall be completely removed prior to application of coating. Any cleaning procedure that safety and thoroughly performs this cleaning without undercutting exposed aggregate will be acceptable. However, the procedure used will be subject to the approval of the Contracting Officer after field demonstration. Some possible cleaning procedures are high-pressure waterjets, air-water cutting, sandblasting, mechanical brushes, or a combination of these techniques...."

250. A water wash with questionable pressure was used to prepare the concrete surface. In some areas this type of preparation worked satisfactorily, but in other areas the prepared surface had loose friable mortar between large aggregate particles which could easily be removed by hand or with a screwdriver. As unfortunately is the case in many construction projects, contract administration and scheduling problems did not permit proper cleanup of all areas prior to application of the new materials.

251. Six shotcrete mixtures were used in the demonstration as follows:

Mixture No.	Description
1	Conventional shotcrete, Type III cement
2	Conventional shotcrete, Type IA cement
3	Glass fiber-reinforced shotcrete, Type I cement
4	Glass fiber-reinforced shotcrete, Type IA cement
5	Latex-modified, glass fiber-reinforced shotcrete, Type I cement
6	Latex-modified, glass fiber-reinforced shotcrete, Type IA cement

In all of the mixtures, materials were batched and blended at the top of the lock and then brought to the work platform at the application location. Mixtures 1, 3, and 4 were applied from a hanging platform of limited width. The resultant congested working conditions did not allow the nozzleman to follow good practice in shotcrete application. Mixtures 2, 5, and 6 were applied from a much larger floating barge.

252. The conventional shotcrete was applied in a single layer using the dry-mix process. The glass fiber-reinforced shotcrete was applied using the standard "spray-up" process which applies a wet mixture at low pressure, while the fiberglass fibers are chopped into 4-in. lengths and blown against the surface simultaneously with the mortar (Figure 109). The thickness of material applied was only about 1/8 in. per pass, and the surface was rolled with what looked like a serrated paint roller between passes. This roller pressed the glass fibers into close contact with the mortar. A mat of material resulted which could actually be lifted off in sheets, but which also would sag if too much weight was added too fast. All mixtures were applied to the lock

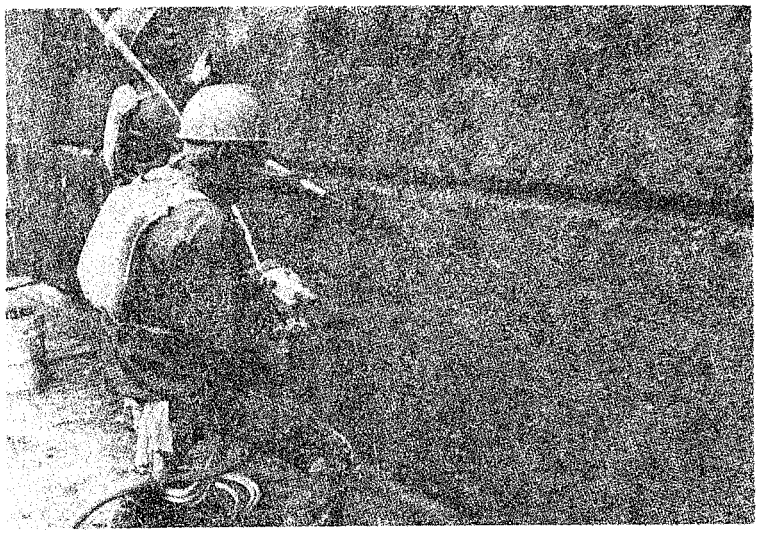


Figure 109. Application of latex-modified, glass fiber-reinforced shotcrete, Lower Monumental Lock

wall at a design thickness of 3/8 in. Field cast test specimens were shot against rigid plywood boards.

253. Evaluation of the various mixtures and application procedures consisted of three basic phases: (a) determining the practicality and speed of application of the coatings using construction crews under true field conditions; (b) extensive laboratory evaluation of test panels made and cured in the field; and (c) evaluation of physical performance of the in-place material after one full year of service.

a. The field applications showed that with experienced crews and proper planning, any of the coating materials could be applied at a reasonable rate under difficult field conditions. From a practical standpoint, the demonstration showed that the latex mixtures had a natural advantage over the conventional mixtures because their use permits a drying period instead of moist cure after application. Good moist curing or properly applied and protected curing compound is typically difficult to obtain in the field.

b. The field cast specimens were trimmed to a thickness of 5/16 in. for testing so that they would all be of the same thickness. Selected material properties for typical panels made in the field were as follows:

Property	Mixture No. 1	Mixture No. 5
Unit weight, pcf	148	142
Air content, %	2.8	1.8
Absorption, %	9	10

(Continued)

Property	Mixture No. 1	Mixture No. 5
Flexural strength, psi	890	2,770
Tensile strength, psi	225	780
Impact resistance, blows	1	500*
Resistance to cycles of freezing and thawing, % weight loss (cycles)	31 (196)	1(345)
Coefficient of permeability at 100 ft of head, ft/min/ft head x 10^{-9}	18,400	1

* Essentially no damage, test stopped.

c. A portion of the test section nine months after the repair is shown in Figure 110. One year following completion of the trial repair the test sections were inspected to determine the condition of the various shotcrete mixtures. The condition of each section was described as follows:

(1) Mixture 1. Hollow sounding (debonded) areas were present on most lift sections. The sizes of the debonded areas varied considerably, ranging from a few inches in diameter to nearly the entire lift section. A number of fine cracks were present. Although debonded areas and fine cracks were present, the coating surface itself was

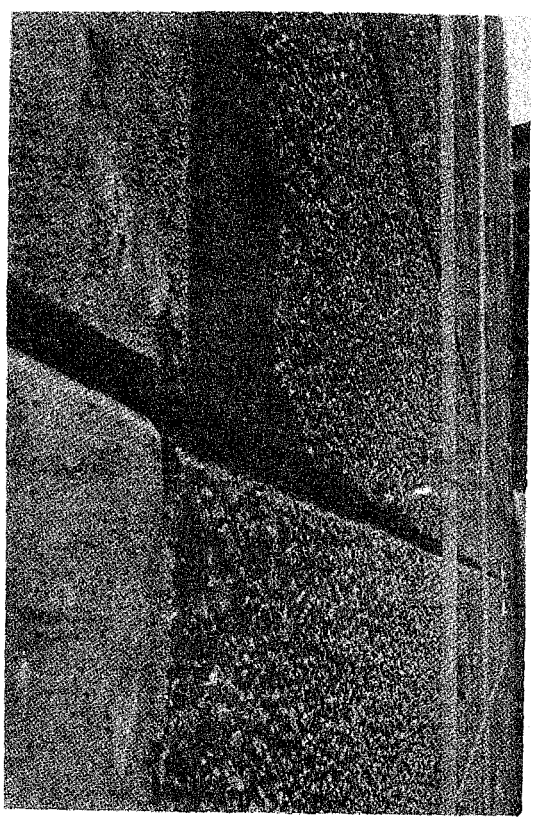


Figure 110. Comparison of coated (left) and uncoated areas of wall, Lower Monumental Lock

generally sound, with essentially no change in appearance (with the exception of fine cracks) from the previous summer.

(2) Mixture 2. The fine cracks evident here did not appear to be as numerous as in Mixture 1. Also there appeared to be fewer hollow sounding areas as compared to Mixture 1. In this section, a large crack in the monolith was noted during application of the coating. This crack reflected through the coating as one tight crack, with no spalling around it.

(3) Mixture 3. No fine cracks were evident in this section, and very few debonded areas were present. It should be noted that the preparation of the surfaces prior to application of coatings the previous summer was observed to be better on sections where Mixtures 3 and 4 were applied, as compared to the other four sections. The observations after one year of field service help to confirm the importance of good surface preparation. The surface of this coating showed essentially no change in appearance from the previous summer.

(4) Mixture 4. A few fine cracks were noted in only one area. The overall percentage of debonded areas on this section was very low. The surface appearance was essentially unchanged from the previous summer.

(5) Mixture 5. No fine cracks were evident in this section. No hollow sounding (debonded) areas were evident on the upper lifts. However, some of the lower lift sections were almost completely debonded. After one year of field service, this section was not as good with respect to bonding to the substrate, as were the sections with glass fibers alone. However, this condition was attributed entirely to differences in surface preparation prior to coating.

(6) Mixture 6. No fine cracks were observed in this section. No debonded areas were evident on the upper lifts, but numerous hollow sounding areas were present in lower lift sections, with some sections having completely peeled off in large sheets. However, immediately next to some of the peeled-off areas, the bond of the coating to the substrate was excellent. As with all other sections, the surfaces of sections coated with Mixtures 5 and 6 showed essentially no change in appearance from the previous summer. Visual examinations of cores (Figure 111) indicated that hollow sounding areas were, in fact, delaminated, and the anticipated well-bonded zones were found to be sound. These observations confirmed that the debonding failures were caused by improper surface preparation. The coatings in such instances were found to be bonded to the mortar from the original (unsound) surface, but the weak surface mortar had peeled away from the rest of the concrete, thereby causing the failure.

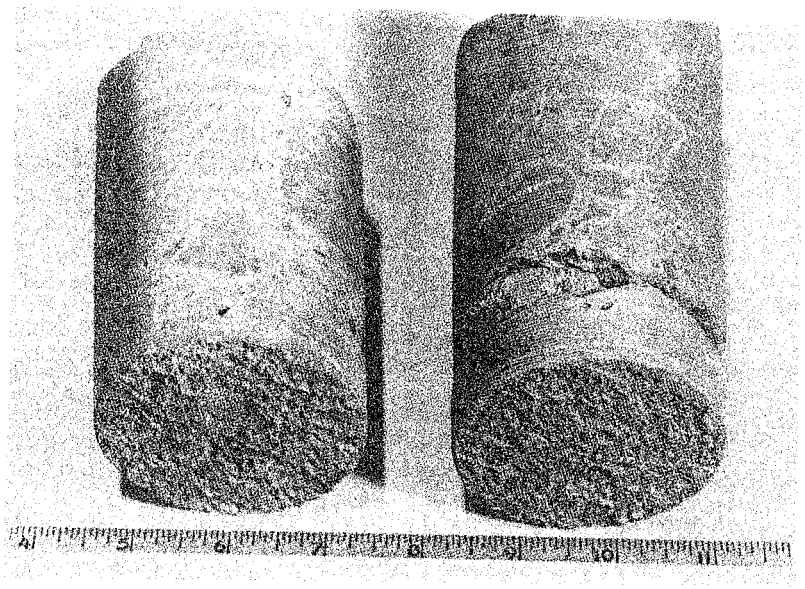


Figure 111. Typical horizontal cores from test sections showing excellent bond (left) and bond failure at shotcrete-concrete interface (right) due to improper surface preparation, Lower Monumental Lock

254. Based on North Pacific Division laboratory tests and field trials, it was concluded that thin shotcrete coatings such as the fiber-reinforced, latex-modified system should be extremely effective in terms of time and cost savings when compared to the conventional alternative of concrete removal and replacement. Tests on field cast panels indicated that:

a. Overall, the best performance can be achieved by a combination of glass fibers and latex, followed in order by glass fibers alone and by conventional shotcrete. The superiority of the latex mixture was especially apparent in the permeability and freeze-thaw tests.

b. Proper surface preparations are absolutely essential prior to application of any coating.

c. Air-entraining cement did not produce adequate entrained air in any of the mixtures used, regardless of mixer type and regardless of whether the wet-mix or dry-mix process was used.

d. In determining which coating materials to use in future lock wall repairs, a careful cost/benefit evaluation should be made. Conventional shotcrete will likely give a few to several years of good service. The glass fiber and glass fiber latex coatings will give better service for successively longer periods of time, but at respectively greater initial cost.

255. Based on the field and laboratory evaluations, it was decided to apply a 3/8-in. thick, fiberglass-reinforced, latex-modified cement coating to

the lock chamber walls. Since the success of the repair was totally dependent upon satisfactory surface preparation, several additional surface cleaning trials were made to verify that with properly operating high-pressure water-jet equipment, the surface could be satisfactorily cleaned. Sandblasting equipment, wire brushes, hand chipping, and air-operated scabblers were also tried by the Government designers. The benefit of this experience was passed on to the bidders through a prebid conference, discussion in the technical provisions of the bidding documents, and with photographs of the work (Figure 112) which also became a part of the bidding documents.

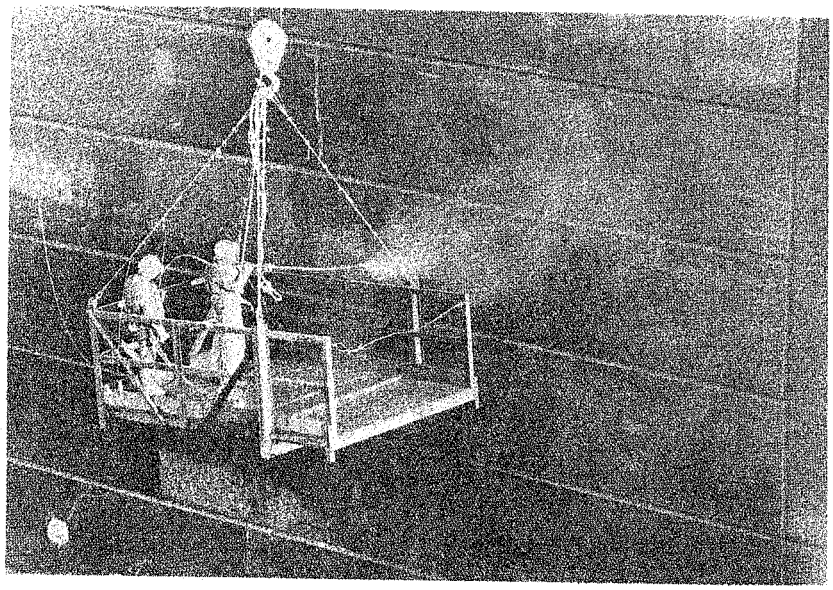
256. A prebid conference which included a question-answer period and open dialogue between bidders and the designer, inspection crew, and operations personnel from the project was held. The designer explained what the concrete problem was and what this repair needed to accomplish. Slides of the previous year's trials and the surface cleaning procedures were shown. Contractural problems experienced during the trials were discussed. After the conference, a tour through the lock was conducted. Ten contractors attended the conference, including the successful low bidder. The prebid conference was probably one of the reasons for the success of the job and the lack of claims and modifications.

257. The Government estimate and three bids were as follows:

	Mobilization	Surface Preparation and Coating	Total
Government	$145,600	$ 885,000	$1,030,600
Bidder 1	437,000	742,057	1,179,057
Bidder 2	419,000	933,000	1,352,000
Bidder 3	900,000	1,046,000	1,946,000

The Government estimate was originally $937,000 total, or 25.8 percent below the low bidder. A review of the estimate found errors that increased it to $1,030,600 or well within 25 percent of the low bidder which was a criterion for award of the contract. Early in 1980, a construction contract for the repairs was awarded to the low bidder, Premier Waterproofing Company, Denver, Colorado.

258. There was no device or test which could be used to measure the degree of "clean" necessary and actually achieved during surface preparation. To minimize disagreements and the application of different standards by



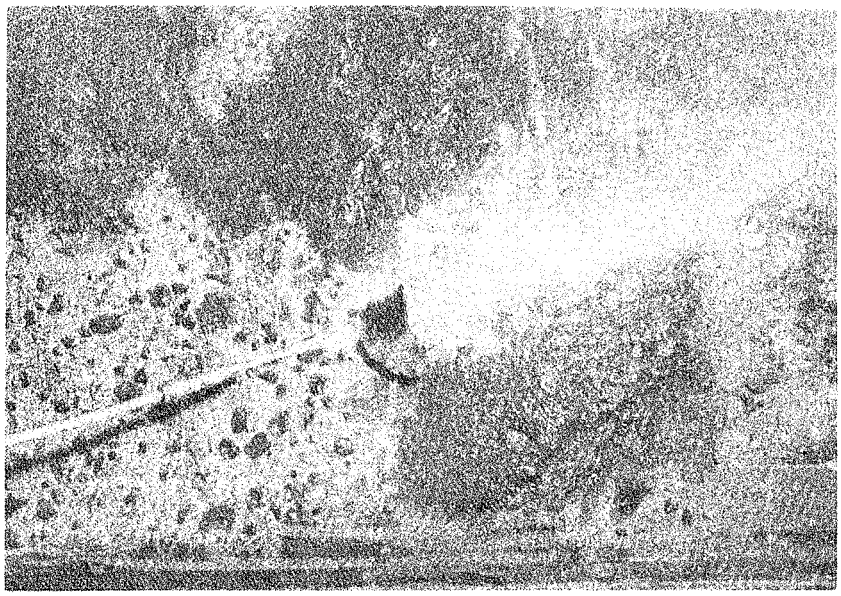


Figure 112. Removal of unsound concrete mortar using a high-pressure water jet, Lower Monumental Lock

different inspectors and to establish with the contractor right from the start what would be acceptable, the contract required preparing "sample areas" before progressing with cleanup of the entire lock. The sample areas showed minimal, medium, and serious deterioration. The contractor's foreman, field supervisors, and company owner were there. The Corps had its designer, shift supervisors for inspection, and head of the construction administration division present. The equipment to be used in the actual cleaning was used in the demonstration. Agreement was easily reached as to what was required and what was an acceptable condition. This standard was then used throughout the job.

259. Most of the actual cleaning was done between lockages over a three-week period before the lock outage and actual spray-up coating operations began. High-pressure water-jet equipment was used and normally operated at about 10,000 psi. As the nozzle tips would wear out, the pressure would start to drop to about 7,000 psi. New tips were then installed. Both the pressure and angle of the jet were critical to effectively remove unsound materials. Occasional handchipping was used to supplement the water jet in areas where it just did not clean well enough. Usually the unsound mortar was about 1/4- to 3/8-in. thick and flaked off easily (Figure 113). For most of the area in the lower portion of the lock, scaling had already occurred to a depth of 1/2 to 2 in. in the mortar, severely exposing the large aggregate which itself was quite sound. An example of the wall surface after acceptable surface preparation is shown in Figure 114.

260. The contractor was experienced with various forms of specialty concrete construction but had no experience in the spray-up process. The specifications clearly required various degrees of experience for the nozzlemen and actual applicators. The foremen were required to have had at least two years experience with shotcrete, and at least two of the nozzlemen were required to have served at least 6-months apprenticeship with the same type of equipment used on the job. All other nozzlemen were required to have had at least two weeks of "hands-on" training. Each spray-up crew was required to demonstrate their ability to perform satisfactorily and to apply coatings of the required quality by actual placements.

261. The contractor met these requirements in two ways. First, he hired all available experienced spray-up crews from two companies that specialize in this work. Second, he set up a training center in a warehouse for his own personnel and had them practice for two weeks before going to the

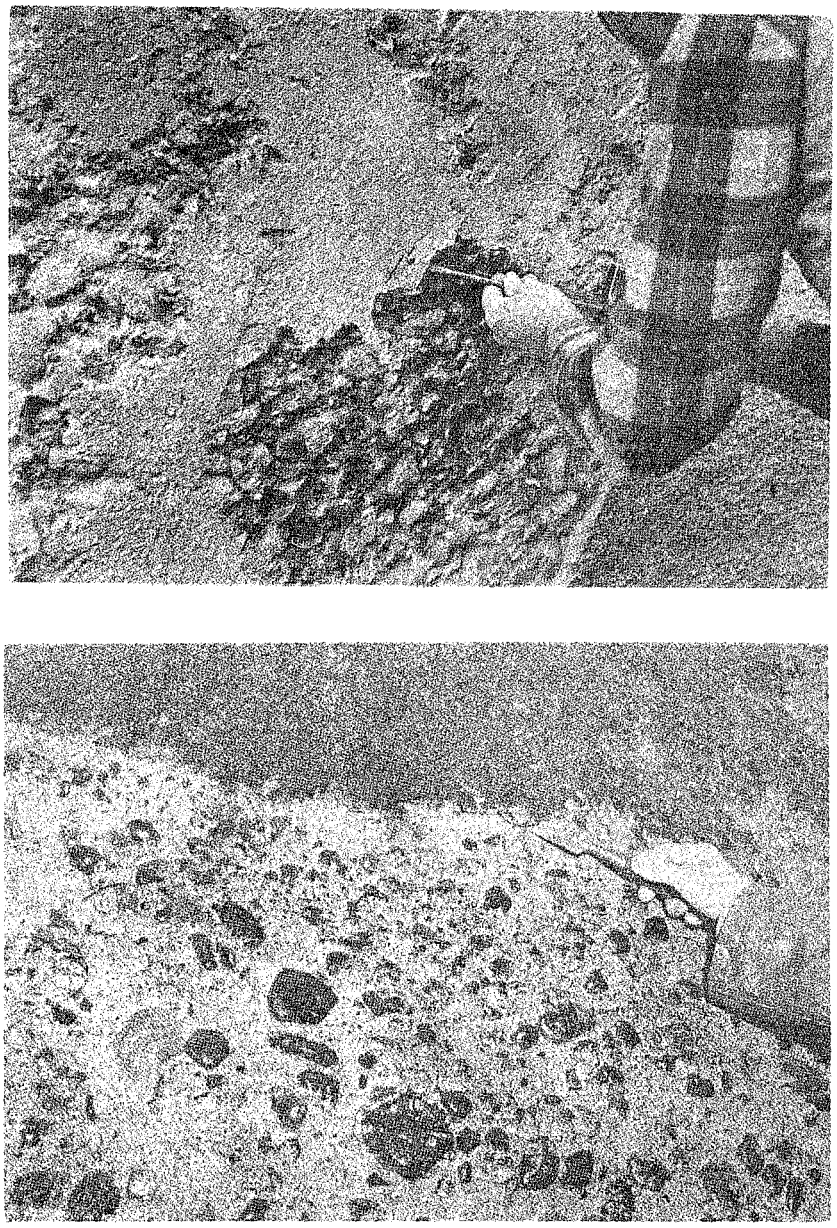


Figure 113. Examples of how easily the unsound surface mortar could be removed, Lower Monumental Lock

Figure 114. Typical condition of the wall following acceptable surface preparation, Lower Monumental Lock

jobsite. During this training, a knowledgeable factory representative for the fiberglass supplier gave "hands-on" instruction, and one of the experienced crews worked with the new crews for about two days. After meeting the minimum experience requirements, each nozzleman made a sample panel from which test coupons were cut. These coupons were examined visually for laminations, porosity, fiber distribution, and appearance. They were then measured for thickness and subjected to center-point flexural testing.

262. The mixture used for all trials was the same as specified and used throughout the contract work. It was identical to the latex-modified, glass fiber-reinforced, sprayed-on mortar coating (Mixture 5) used in the field demonstrations of the previous year. The batch weights and volumes were as follows:

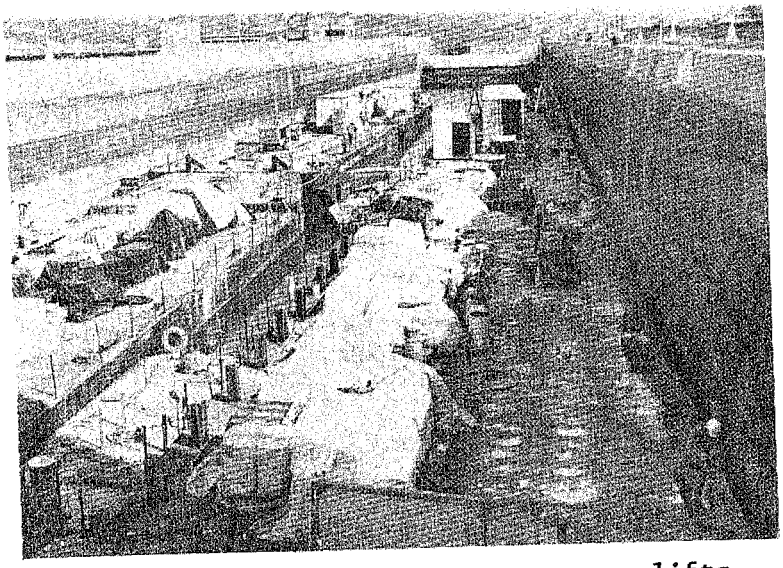
Item	Weight, lb/cu yd	Approximate Volume, cu ft
Water	245	4
Cement	1,720	9
Fibers	117	1
Latex	520	7
Fine aggregate (SSD)	860	5
Water-reducing admixture	75	--
Air (3.7 percent)	--	1
		27

The cement used was portland cement Type I-II. The fibers were single-strand, multiple-filament, alkali-resistant fiberglass. The latex contained approximately 50 percent solids and an antifoaming additive. The fine aggregate was presacked sand. The water-reducing agent met the applicable requirements of ASTM C 494.

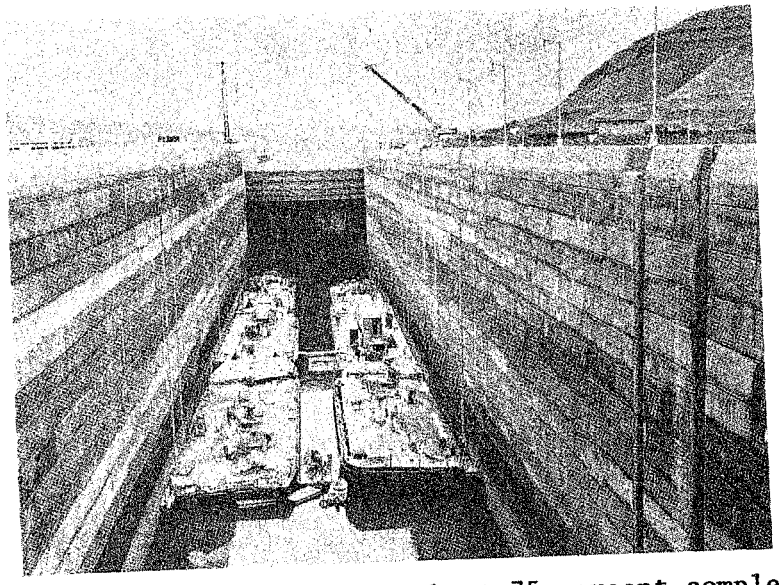
263. One week before the lock was taken out of service for the actual repairs, the contractor was required to go through a full-scale demonstration of his operation during a 24-hr lock shutdown. The demonstration required cleaning and coating an upper lift of the lock wall. It gave the contractor and crews a chance to find out ahead of time what problems might be encountered during the actual project shutdown and gave them time to react to the day's experience before getting into the three-week-outage period.

264. The contractor chose to work from a floating plant consisting of six barges, three working at each wall and lashed together end-to-end. The barges stretched for about 75 percent of the length of the lock. The lashed barges were braced and welded side-by-side so that they all acted as one unit. At the start of the lock outage, the barges were floated into the lock chamber and positioned. The lock was filled and the contractor began working from the top lift down (Figure 115). After the approximate height of one 5-ft lift was coated from end-to-end on each side, the lock level was lowered about 5 ft and the next lift was coated.

265. Each spray-up station (Figure 116) had its own scales, spray-up gear or nozzle, mixer, and pump. Each station was also manned by a full crew consisting of a nozzleman, mixer man, roller man, and usually a helper. As previously described the "spray-up" process used a wet mortar mixture pumped to the nozzle by grout pump and atomized with low-pressure air at the nozzle. While the mortar was sprayed against the wall surface, fiberglass fibers were simultaneously chopped from a continuous strand and blown into the mortar spray from a separate cutter head attached to the nozzle. The material was applied in a thickness of about 1/8 in. per pass and was lightly rolled with what looked like a serrated paint roller between successive passes. This roller pressed the glass fiber into intimate contact with the mortar. A mat of material resulted which was heavily reinforced but which would sag or fall off if too much weight was added too fast. The final surface could be troweled; but although this process made a better appearance, it was unnecessary and if overdone could have been damaging.

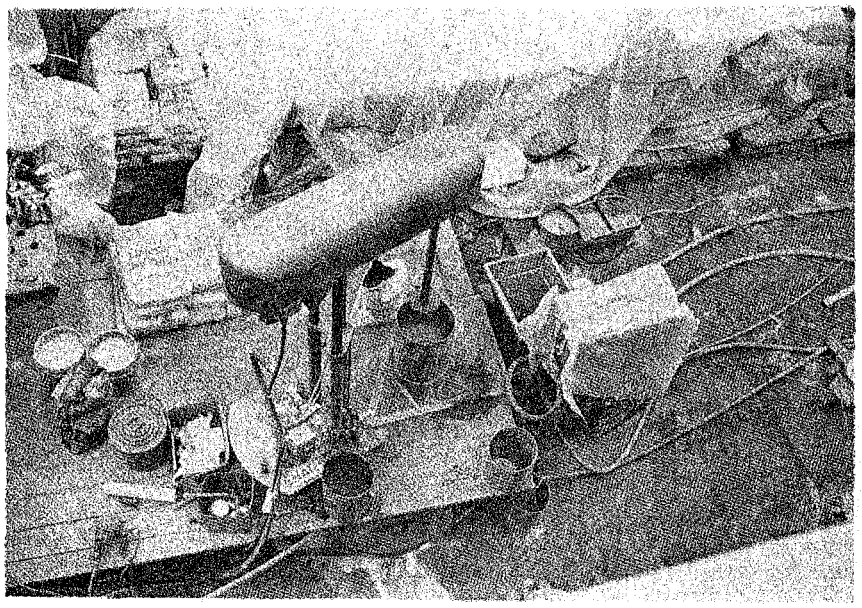


a. Coating application near upper lifts

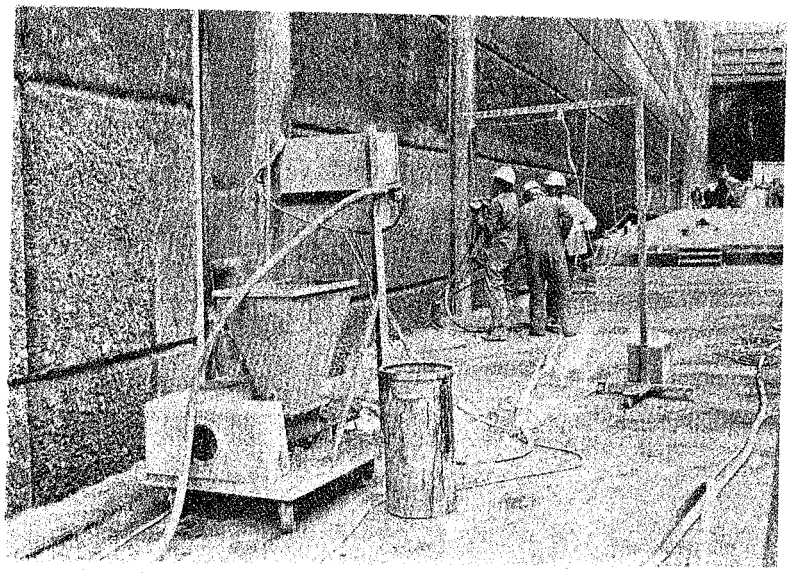


b. Coating application about 75 percent complete

Figure 115. Floating plant used in coating application, Lower Monumental Lock



a. High-shear mortar mix



b. Group pump and spray-up unit

Figure 116. Typical "spray-up" station, Lower Monumental Lock

266. As the work progressed from the upper lifts which had less deterioration (Figure 117) to the lower lifts with substantial deterioration, the length of fiber used had to be adjusted. The cutter heads were designed to chop fibers of 1-1/2-, 4-1/2-, or a combination of 1-1/2- with 3-in. lengths. The longer fibers were desired because of their believed tendency to give greater toughness and strain capacity. However, where the depth of relief around large exposed aggregate was about 1/2 in. or more, the long fibers tended to bridge across adjacent pieces of aggregate leaving an air void over the relief between them. Where a combination of 1-1/2- with 3-in. fibers was used, this problem was minimized.

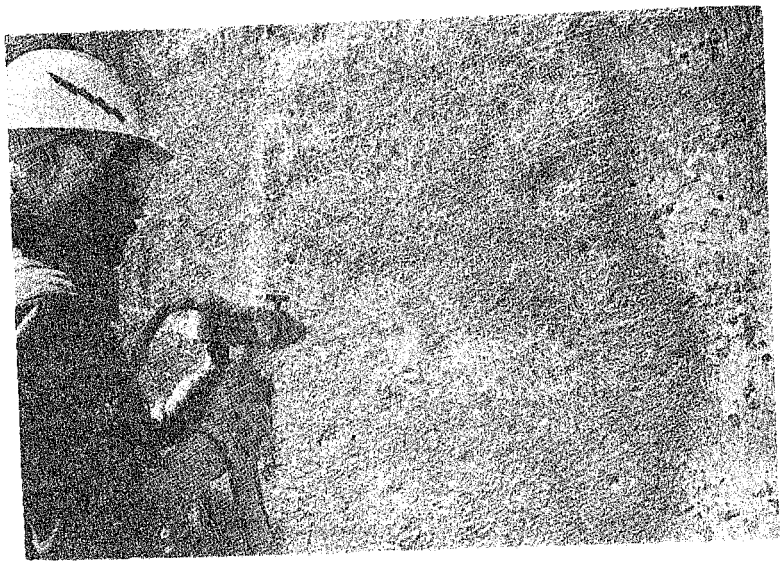


Figure 117. Coating application in an area of minimal scaling near the top of the lock, Lower Monumental Lock

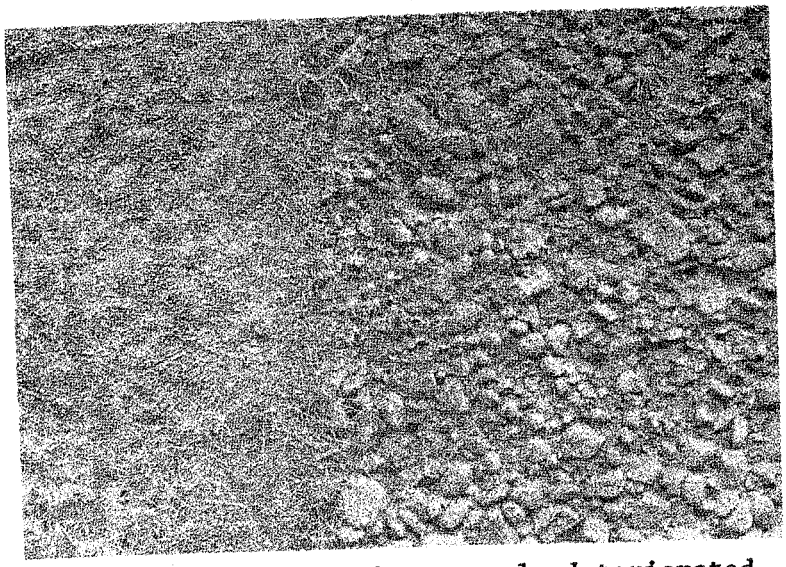
267. Horizontal joints were treated by simply spraying over them and treating them as a continuous mass. These joints have no known movement across them and are tight construction lift joints. They did present some problem because they had a 3-in. chamfer into which the coating had to be sprayed and rolled. There was a tendency for the workmen not to adequately coat or roll these places. Vertical joints between monoliths were known to have measurable amounts of relative movements. Wherever these movements were encountered, the joint was oversprayed from one side to the other, but then the joint line was cut through the coating with a knife while it was still wet. Some joints that were not cut with the knife were later cut with a diamond saw.

268. The thickness of the in-place coating (Figure 118) varied between 5/16 and 5/8 in. and averaged 3/8 in. In areas where 3-in. aggregate was visible on the lock wall, the thickness over the outside of the 3-in. cobbles was less than 3/8 in., but between the aggregate particles it was more than 3/8 in. Since protection over the mortar between the aggregate was most important, this thickness was considered acceptable. Thickness was continuously checked by stabbing the wet mix with a nail used as a depth gage.

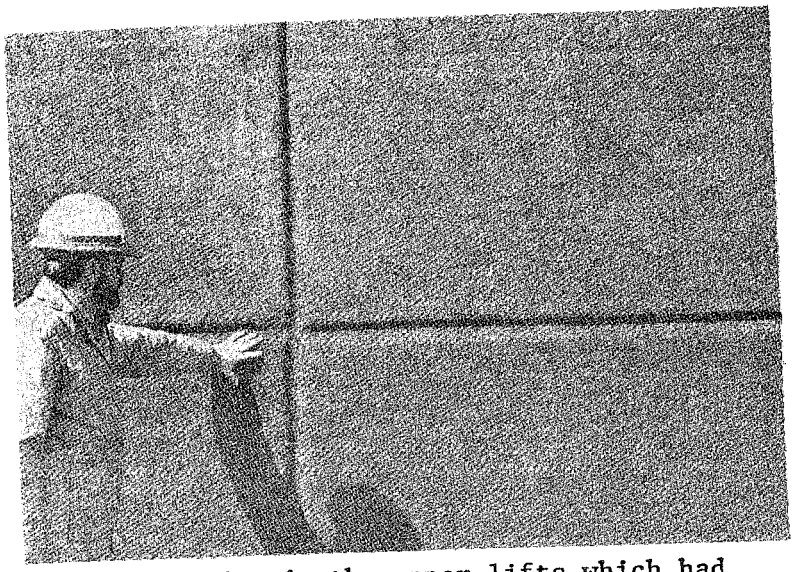
269. In order to coat the wall below minimum tailwater, it was necessary to set the downstream lock chamber bulkhead and partially unwater the lock, so the barge floated below tailwater. After completion of the spray-up work, the water level was kept below the level of the last coating until it air dried for 36 hr as required by the specifications. In actuality, the contractor finished about one day earlier than necessary, and the coating was able to receive a little bit more beneficial drying before being inundated by reflooding of the lock chamber to full tailwater elevation. The job was completed on time with no accidents, no claims, and no modifications.

270. After about six months in service, the coating appeared to be in good condition as shown in Figure 119. During the June 1981 lock outage, approximately one year after the coating was applied, the lock walls were evaluated by soundings, visual examination, and core drilling. Based on soundings at a typical 45-ft wide by 100-ft high section of monolith 15, it was concluded that 99.2 percent of the coating was fully bonded. About 20 unbonded areas were found. Generally, the unbonded areas were about 1 sq ft or less in area and located just below a lift joint or adjacent to a monolith joint. These unbonded areas were attributed to the fact that the workmen who applied the coating had a natural tendency to underspray or underroll near lift joints. Also, because of the nature of the coating material, it usually would pull loose or slough near monolith joints when it was trimmed to maintain the joint line. It would then be rerolled as best as was practical, but apparently this procedure was not totally effective.

271. Forty cores were taken through the coating and into the base concrete in monoliths 15 and 16. In each monolith, cores were taken along a vertical line near the upstream monolith joint and the third point which was the approximate center of an uncracked mass. Along each vertical line a core was generally taken in the center of the first 5-ft lift and at the center of

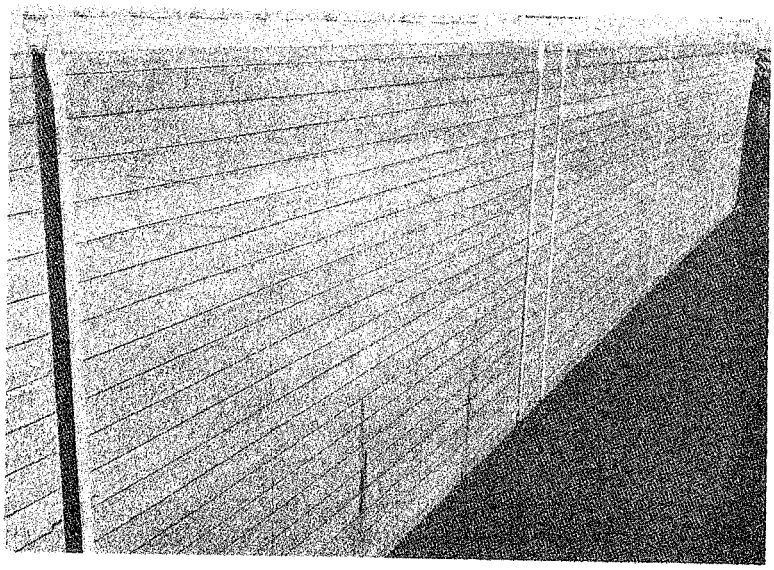


a. Coating of rough, severely deteriorated concrete

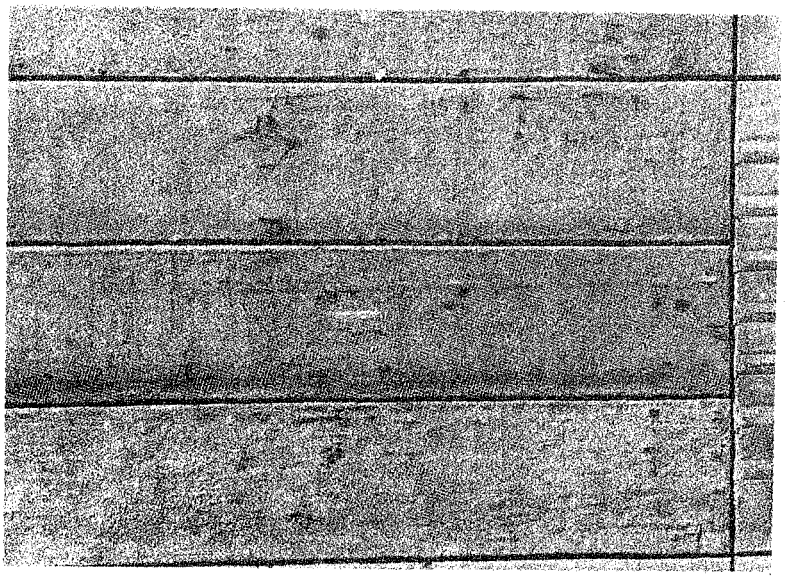


b. Coating in the upper lifts which had minimal concrete deterioration

Figure 118. Typical appearance of the wall coating, Lower Monumental Lock



a. Overall view of north wall



b. Evidence of barge impact on the coating

Figure 119. Condition of the coating after about six months in service, Lower Monumental Lock

every third lift below it down to tailwater. Midway between the top and bottom lifts, cores were taken along each vertical line 1 ft from the bottom of the lift. Results of an examination of these cores were as follows:

a. The coating thickness ranged from 1/4 to 1/2 in.; however in most cases the thickness was very close to the 3/8-in. design.

b. In most cases the coatings were bonded, very dense, tight, and without serious voids. A few had some small air bubbles where the mortar was not fully densified by rolling, and some interior uncoated glass fibers resulted. However, in no case was there a continuous void or potential seepage path through the coating.

c. In one case the coating broke off as a result of prying the core with a screwdriver to break it loose from the wall after drilling. Failure occurred just beyond the coating-concrete interface in what appeared to be some unsound surface mortar.

d. In the two cases where the coating was not bonded to the concrete, failure was attributed to improper application. There was no evidence of latex mortar being applied before or with the fiberglass fibers when the spray-up process started. A thin layer of clean, white, loose fibers was found between the coating and lock wall.

272. The largest unbonded area found above tailwater was located near the upstream joint in monolith 15. Injection and vent ports were drilled through the coating, and epoxy was injected into the delaminated area in an attempt to bond the coating to the wall. The epoxy seeped through the coating almost immediately as it was being injected under low pressure. Injection was stopped, and the epoxy in the coating hardened. A core through the area showed that essentially no epoxy remained behind the coating and no rebonding occurred.

273. Most, if not all, of the bottom lift of coating, below tailwater, was obviously debonded and some had fallen off the wall. This failure was attributed to either insufficient drying time prior to inundation or re-emulsification of the latex because of continuous saturation. Fortunately, the coating is not considered necessary in this area, nor is it visible during normal operation of the lock.

274. The trial shotcrete coatings applied to monolith 9 in 1979 have performed poorly. Additional failures of some of these coatings had occurred by 1981.

275. Several small, isolated debonded areas of the latex-modified, fiber-reinforced shotcrete coating were removed and resprayed under a

miscellaneous repair contract in March 1983. Also, a fairly large debonded area on monolith 11 and the failed areas of trial coatings on monolith 9 were resprayed. Equipment and procedures, including surface preparation, were essentially the same as those used in the 1980 repairs with the exception that the latex used in 1983 was styrene butadiene instead of saran.

276. In September 1983, it was reported that almost all of the coatings applied in March 1983 had failed. Generally, failures occurred within the concrete substrate immediately behind the concrete-shotcrete interface. The remainder of the lock wall coatings were reported to be in very good condition with the exception of the trial coatings on monolith 9. It was estimated that 40 percent of the trial area had failed. During the inspection, several sheets of the debonded trial coatings were pulled off relatively easily by hand. It was recommended that project personnel remove the remaining debonded areas and an area on monolith 11 to avoid a potential safety hazard.

277. Currently, it is reported that an estimated 15-20 percent of the latex-modified, fiber-reinforced shotcrete applied in 1980 was debonded following the winter of 1985-86 and that a large piece of the coating had fallen off, striking a barge inside the lock chamber.

Emsworth Locks

278. The Emsworth project is located on the Ohio River and consists of two dam structures, one on each side of Neville Island, and two parallel lock chambers. The main channel dam and the locks are 6.2 miles below the confluence of the Allegheny and Monongahela Rivers at Pittsburgh, Pennsylvania. The landside lock is 110 by 600 ft, and the riverside lock is 56 by 360 ft. The lift between lower and upper pool is 18 ft, and the top of the lock walls is 7 ft above upper pool. The chamber walls are concrete gravity structures founded on rock. The structures were originally constructed during the period 1919-1922 and have been operated and maintained since September 1921. A history of major repairs to the locks is shown in Figure 120.

279. The first periodic inspection was conducted in June 1971. At that time the land, middle, and river wall monoliths were described as in poor condition. The concrete in each wall was in an advanced state of deterioration as evidenced by numerous areas of spalling and cracking and signs of extreme weathering. A crack survey was planned for each of the eight gate monoliths,

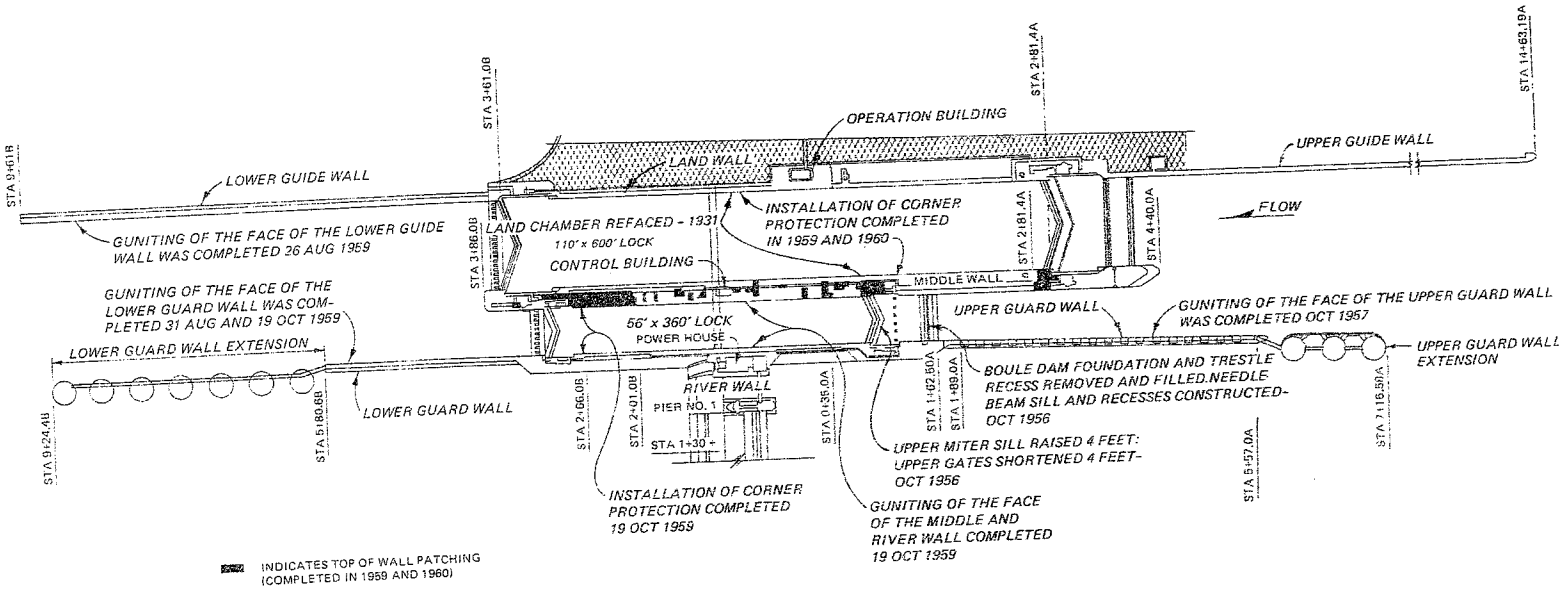


Figure 120. History of major repairs, Emsworth Locks

and contingent upon the availability of funding, it was proposed to drill cores from each gate monolith so that a detailed analysis of concrete condition could be made.

280. The lock wall faces were in poor condition, particularly along vertical joints where significant spalling had occurred. The shotcrete used to reface the river chamber walls in 1959 was deteriorated and in some cases completely missing. The vertical faces of the outer gate recesses had not been refaced and exhibited numerous random surface cracks. The original approach walls were also in poor condition. The upper guide wall extension which was constructed when the dam was rebuilt in 1937 was not as deteriorated as the original approach walls although the concrete was spalled at most of the monolith joints. Shotcrete used to reface the upper guard walls in 1957 was starting to break away from the original concrete. The concrete caps on both guard wall extensions exhibited random cracks, but neither deterioration of the concrete nor loss of cell fill was evident. The esplanade exhibited random cracking, minor spalling and some settlement, none of which was considered serious. Overall, the general appearance of the concrete and shotcrete was poor with numerous areas of severe spalling and disintegration; however there was no evidence of misalignment.

281. Upon completion of the inspection, the inspecting team agreed that the project appeared to be safe under normal operating conditions and that the project was operating satisfactorily. However, it was the consensus of the inspecting team that the normal maintenance program be strictly adhered to and accelerated if necessary in order to keep the project in safe operating condition until replacement or complete rehabilitation could be accomplished.

282. In 1973, the Pittsburgh District initiated an investigation to determine the quality of the concrete in the locks. This study, conducted jointly by WES and the Ohio River Division Laboratory (ORDL), included core drilling and testing, petrographic examination, borehole photography, pulse velocity tests, and a crack survey. A comparison of all laboratory and field tests and core logs indicated that most of the concrete beneath the outer few feet of exterior surfaces was at least of moderate quality with an average compressive strength of about 4,000 psi (Denson and Buck 1974). An examination of the 6-in. diam vertical cores indicated that fragmentation of the concrete near the top of the walls caused by inadequate consolidation and frost action on the nonair-entrained concrete ranged from zero in some cases to a

maximum of 4-1/2 ft. Horizontal cores, usually drilled below the fragmented tops of the walls, exhibited much less damage, generally 1 ft or less.

283. An analysis of the stability of the lock walls (Pace 1976) indicated that, in general, the land wall monoliths did not meet stability criteria for overturning, sliding, or base pressures. Also, some monoliths in the middle and river walls did not meet stability requirements.

284. In November 1980, a demonstration repair was conducted at Emsworth Locks to evaluate the potential of steel fiber-reinforced shotcrete as a repair material. A section of the original upstream guide wall (Figure 121) was cleaned with a high-pressure water jet, and one monolith joint was chipped out in a vee shape to a depth of approximately 6 in. (Figure 122). A strip of joint filler board was placed in the vee joint prior to shotcreting. A dry mix of Fibercrete (sand, cement, and steel fibers) in 60-lb bags was hand-fed into a hopper at a rate of 2 to 3 bags per minute. The 1-in. long steel fibers with hooked ends were 2 percent by weight of the Fibercrete mixture. Water was added at the nozzle via a connection with the local water system. The vee joint was completely filled, and 1 to 2 in. of Fibercrete was placed over the remainder of the test section (Figure 123). Rebound was estimated at less than 10 percent. Although not done for the test section, a striker could be used to shave the wall after it was shotcreted to obtain a smooth surface.

285. According to the supplier, Burrell Construction and Supply Company, Pittsburgh, Pennsylvania, the average placement rate is three bags per minute, and under conditions similar to the test section, 3,000 sq ft per day could be shotcreted with two nozzles in operation. Also, no curing is required unless the temperature exceeds 90°F, in which case hanging wet burlap over the shotcrete for one day would be sufficient. Cost for the in-place shotcrete, as estimated by the supplier, was $4 per sq ft. According to the supplier, flexural and compressive strengths average approximately 900 and 6,000 psi, respectively.

286. A tour of the Emsworth project was made in February 1981 by HQUSACE, Division, District, and WES representatives to examine the condition of the structures, including the Fibercrete test section. After three months in service, the Fibercrete exhibited numerous examples of impact failure, abrasion erosion, and delamination (Figure 124). The explanation for the poor performance of the Fibercrete was that the prepackaged mixture used in the demonstration contained only 60 lb/cu yd of fibers, whereas in the actual

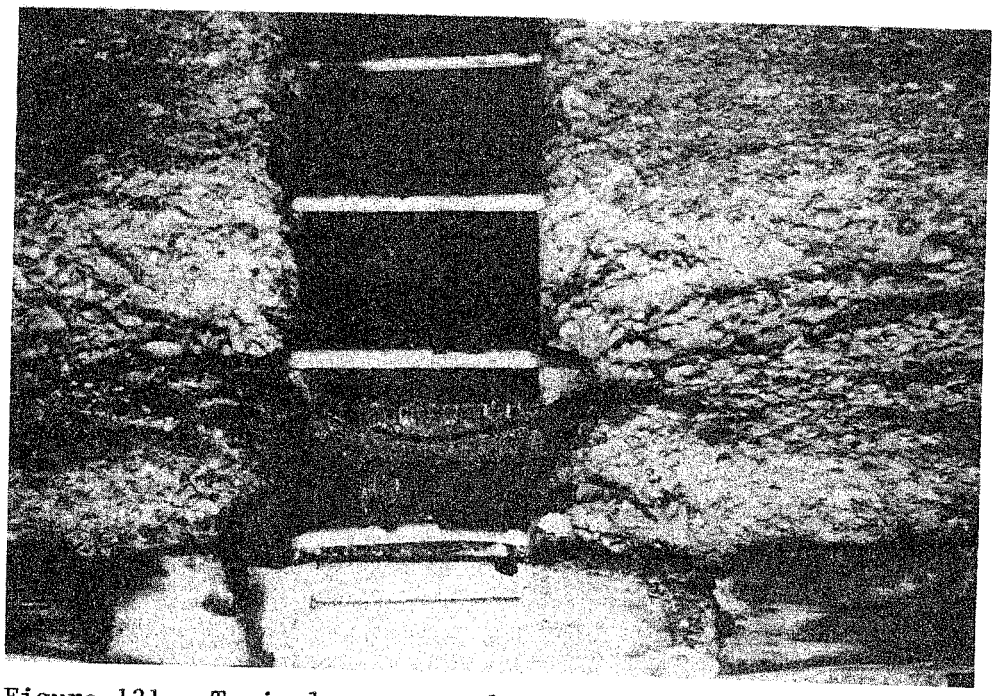


Figure 121. Typical concrete deterioration in upstream guide wall, 1980, Emsworth Locks

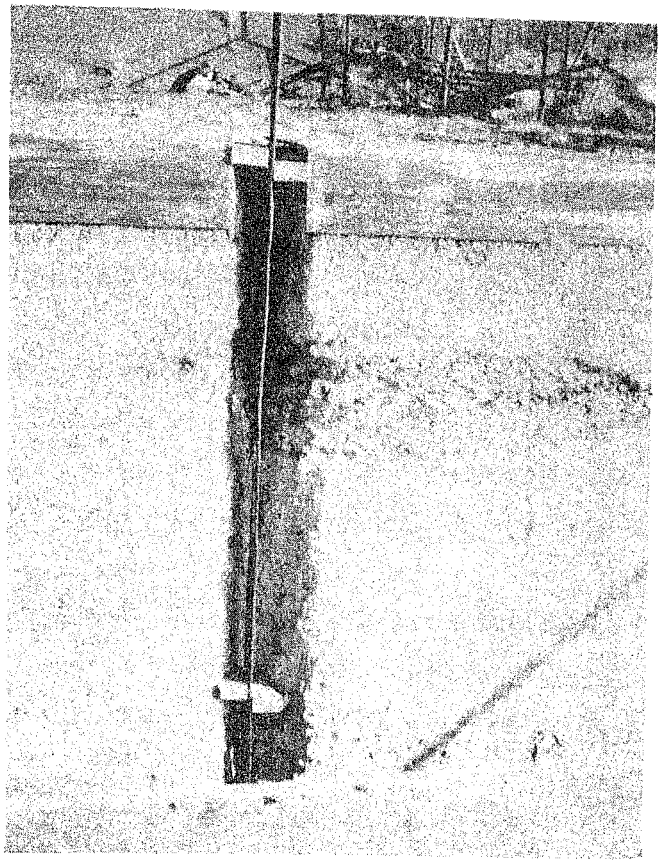
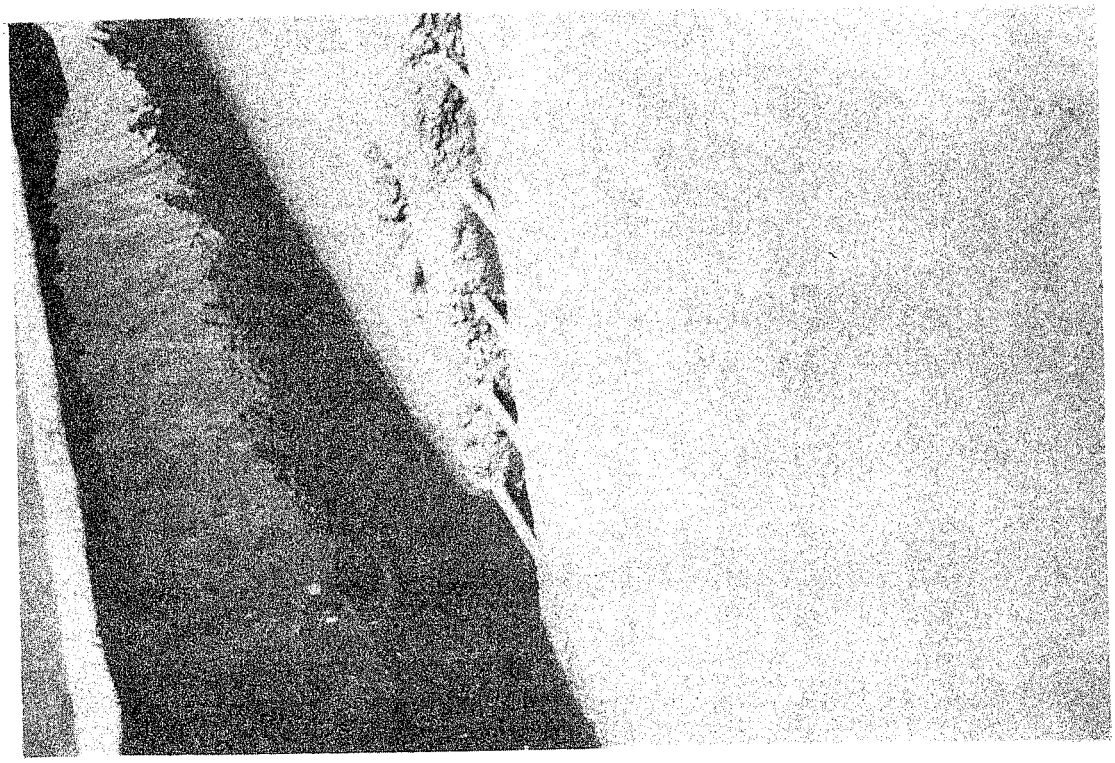


Figure 122. A portion of the prepared test section prior to Fibercrete application, November 1980, Emsworth Locks

a. Application



b. Portion of completed test section

Figure 123. Fibercrete test section, November 1980, Emsworth Locks

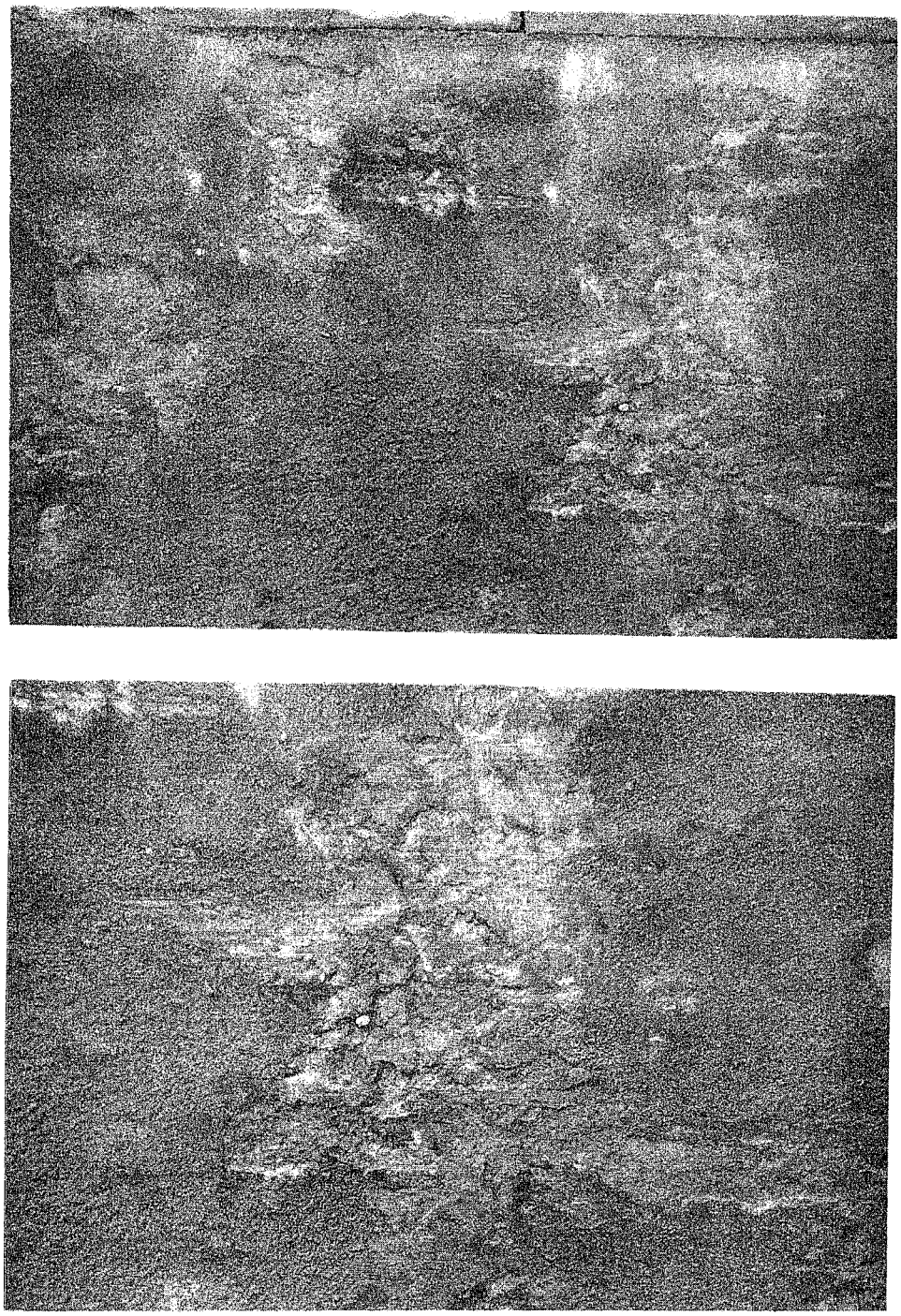


Figure 124. Typical examples of Fibercrete condition, February 1981, Emsworth Locks

repair it was proposed to use a much higher cement content and fiber contents up to 200 lb/cu yd.

287. Considering that the larger land chamber receives most of the heavy barge traffic and that the nonair-entrained concrete had been in service for approximately 50 years, surfaces of the land lock chamber walls appeared to be in relatively good condition (Figure 125) when inspected in 1981. One exception was the monolith joints (Figures 125 and 126). Concrete and shotcrete in the smaller river chamber appeared to be in more advanced stages of deterioration, particularly the land wall (Figure 127) where the smaller tows and pleasure craft using this chamber tie up. Spalling appeared to have originated in the upper portion of the wall where the shotcrete was relatively thin and surface preparation minimal. Spalling propagated down the wall to the point where shotcrete thickness (approximately 4 in.) was sufficient to contain dowels and wire mesh (Figure 128). In comparison, the shotcrete on the river wall of the smaller chamber appeared to be in much better condition (Figure 129). Fifty to seventy percent of the existing shotcrete in the river chamber was reported as "drummy" when sounded.

288. During this inspection, horizontal cores drilled from the lock chamber walls in 1973 were compared. Cores from the river chamber showed the shotcrete to be in generally good condition (Figure 130); however the original concrete behind the shotcrete exhibited significant deterioration probably caused by cycles of freezing and thawing. Cores of similar concrete from the land chamber which were not coated with shotcrete were in generally good condition from the surface inward (Figure 131). These cores appear to be an example of an exterior coating contributing to the saturation of the original concrete with increased deterioration caused by frost action as a result.

289. To ensure operation of the existing locks and dams for another 25 years, plans for rehabilitation were developed (Pittsburgh District 1980). The plan provided for the use of one lock chamber while the other was closed to traffic during rehabilitation. The river chamber would be shutdown for repairs first. All restoration work related to the river chamber operation would be accomplished at this time. Rehabilitation work on the land chamber would begin once the river chamber was placed back in operation. There would be two land chamber shutdowns during this period. The first shutdown would be for 30 days, and the second occurring 12 months later for another 30-day

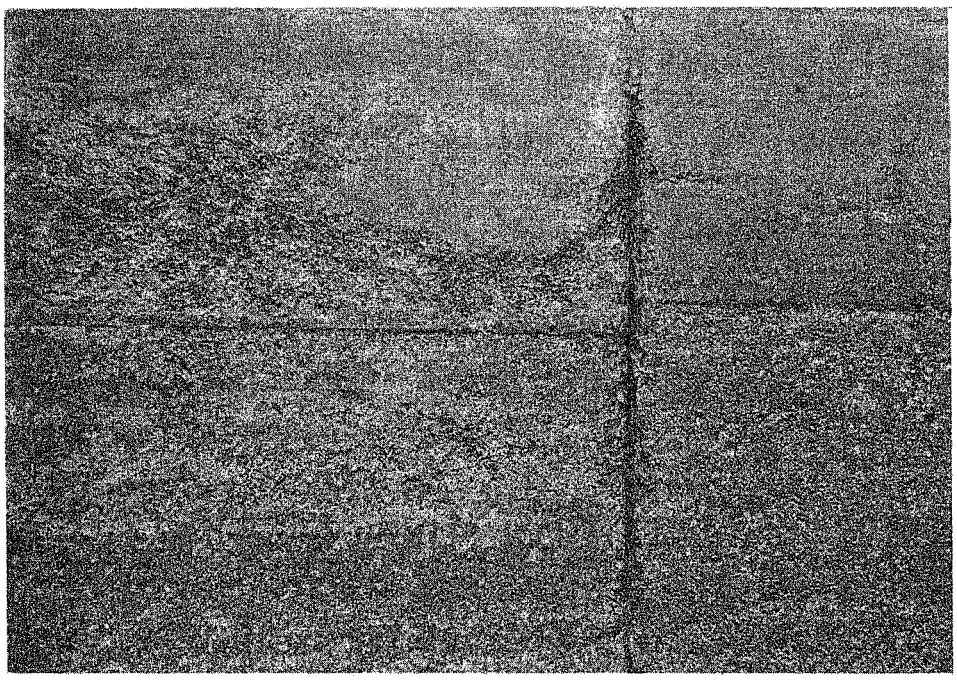


Figure 125. Typical concrete surfaces, land lock chamber wall, 1981, Emsworth Locks

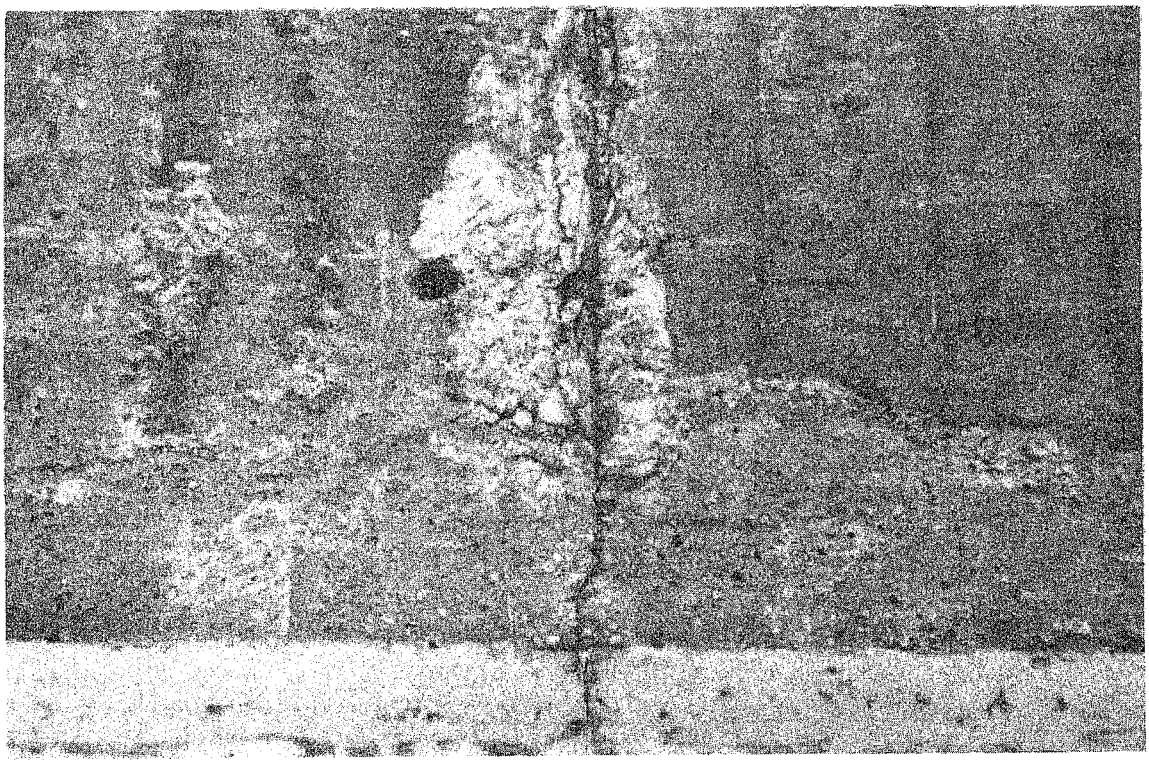


Figure 126. Examples of typical joint deterioration, land lock chamber wall, 1981, Emsworth Locks

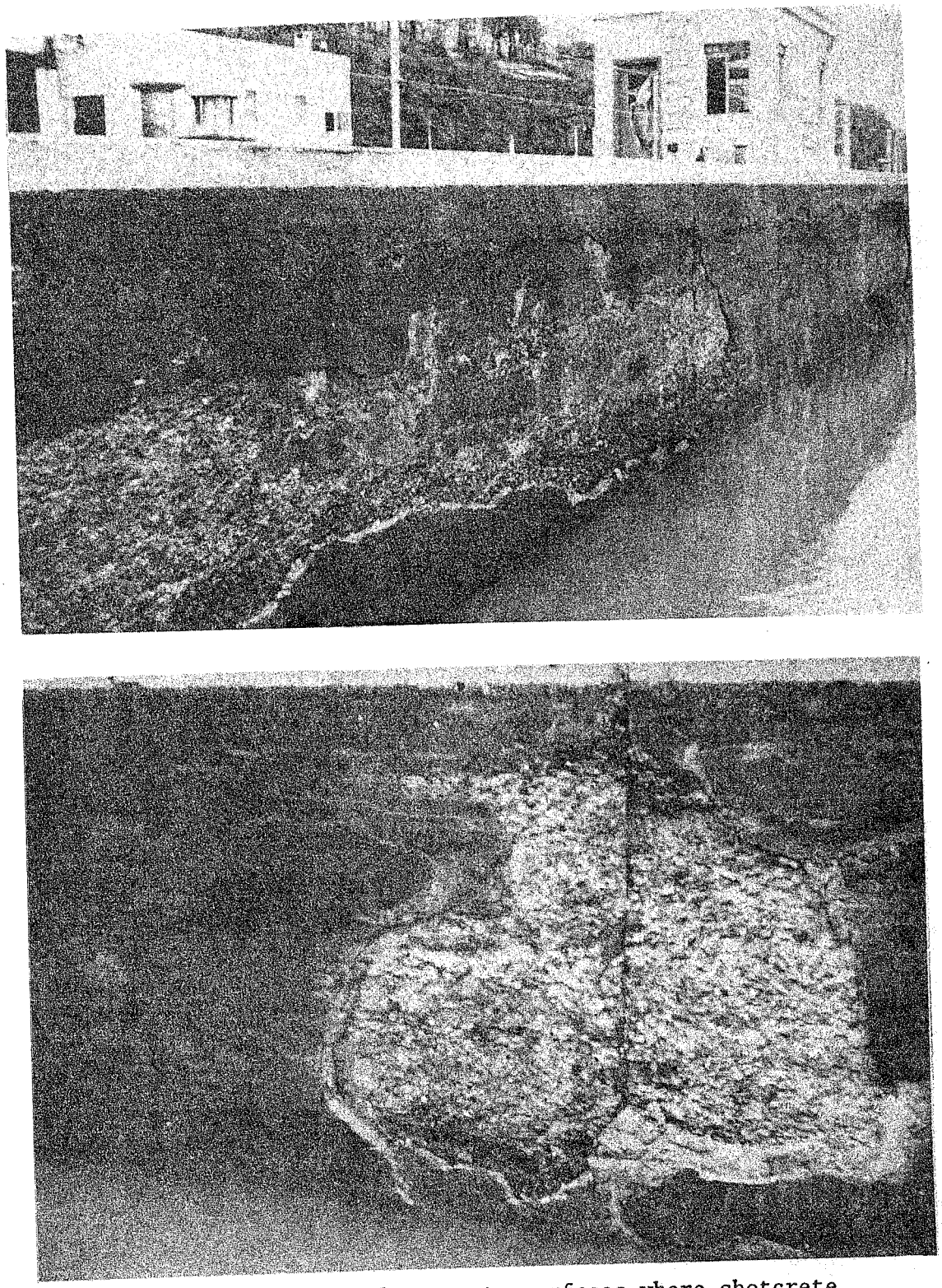


Figure 127. Typical concrete surfaces where shotcrete has spalled on the land wall of the river lock chamber, 1981, Emsworth Locks

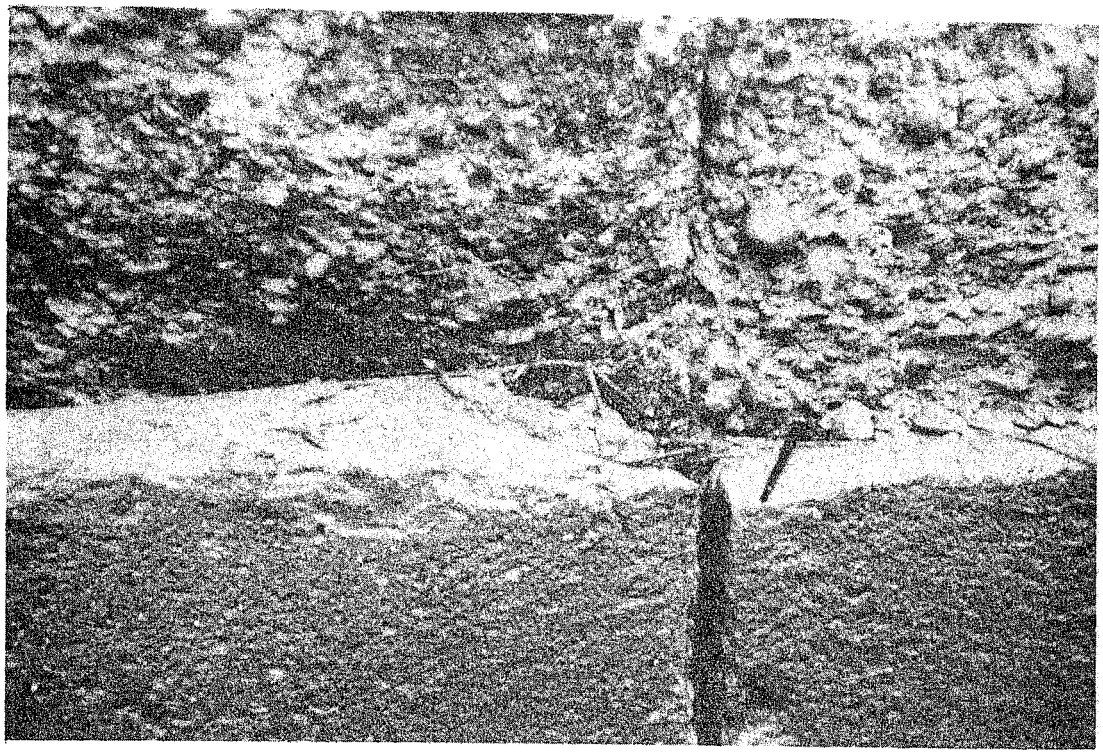


Figure 128. Anchored and reinforced shotcrete remains on the river lock land wall although it was not bonded to the original concrete in several cases, 1981, Emsworth Locks

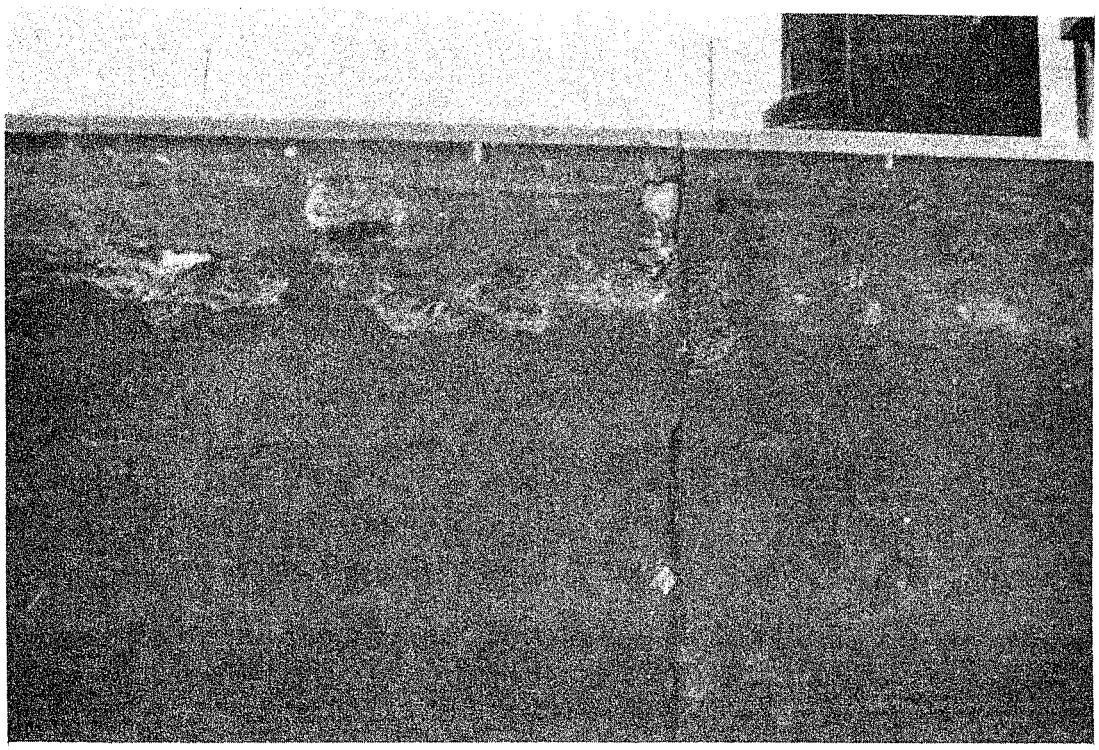
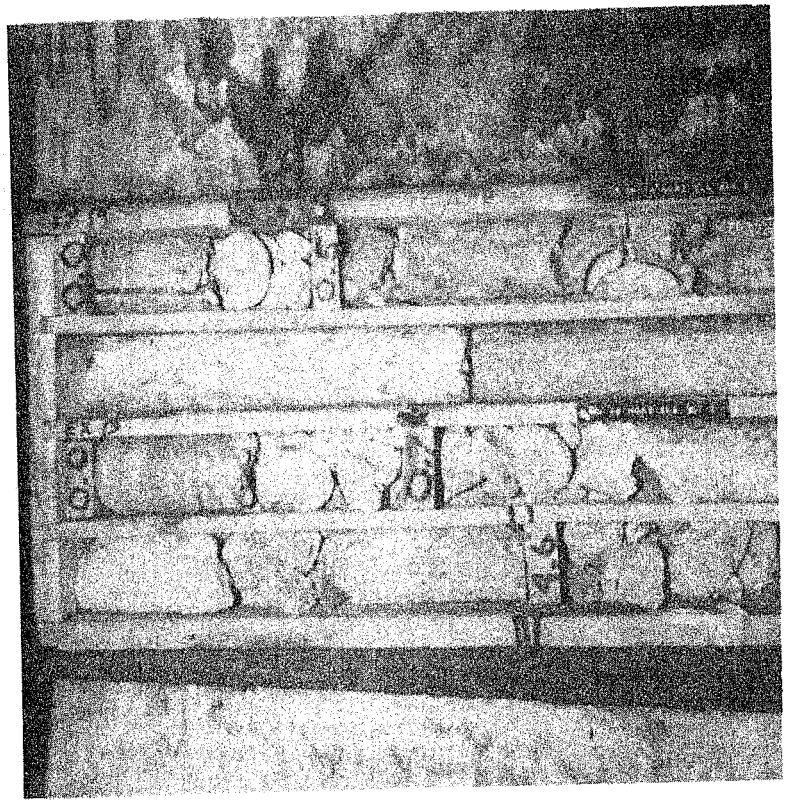


Figure 129. Typical appearance of the river wall in the river lock chamber, 1981, Emsworth Locks

Hole No.
EX-14

Hole No.
EX-13



Hole No.
EX-16

Hole No.
EX-11

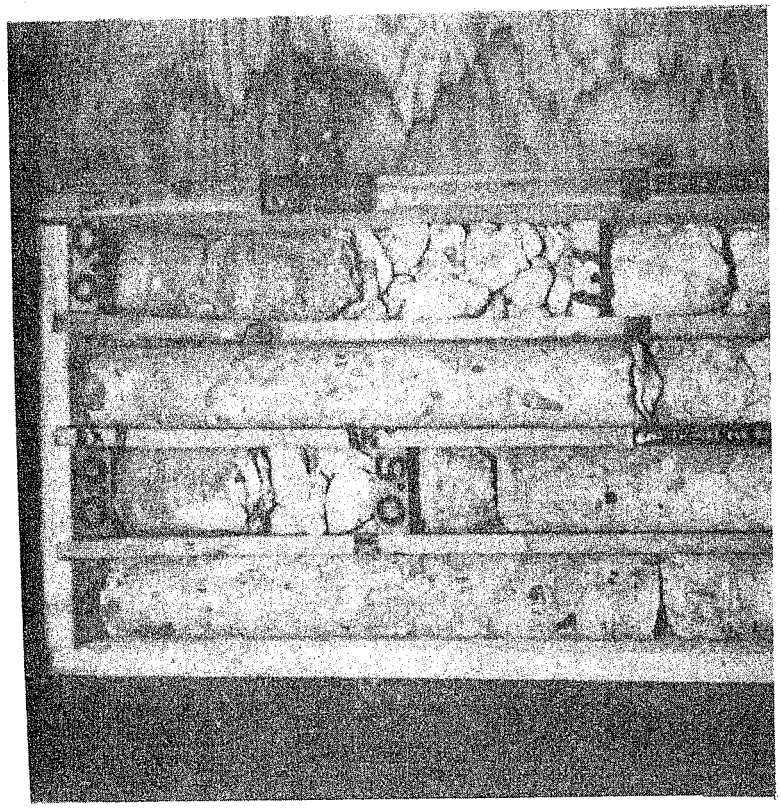
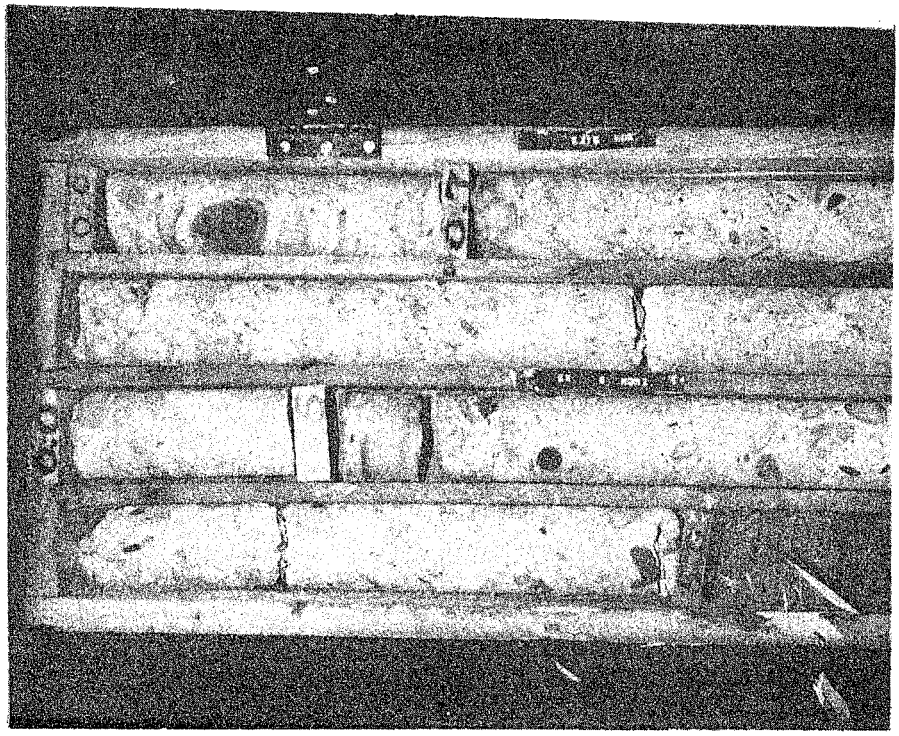


Figure 130. Horizontal cores taken from the river chamber walls, 1973, Emsworth Locks

Hole No.
EX-5

Hole No.
EX-6



Hole No.
EX-8

Hole No.
EX-10

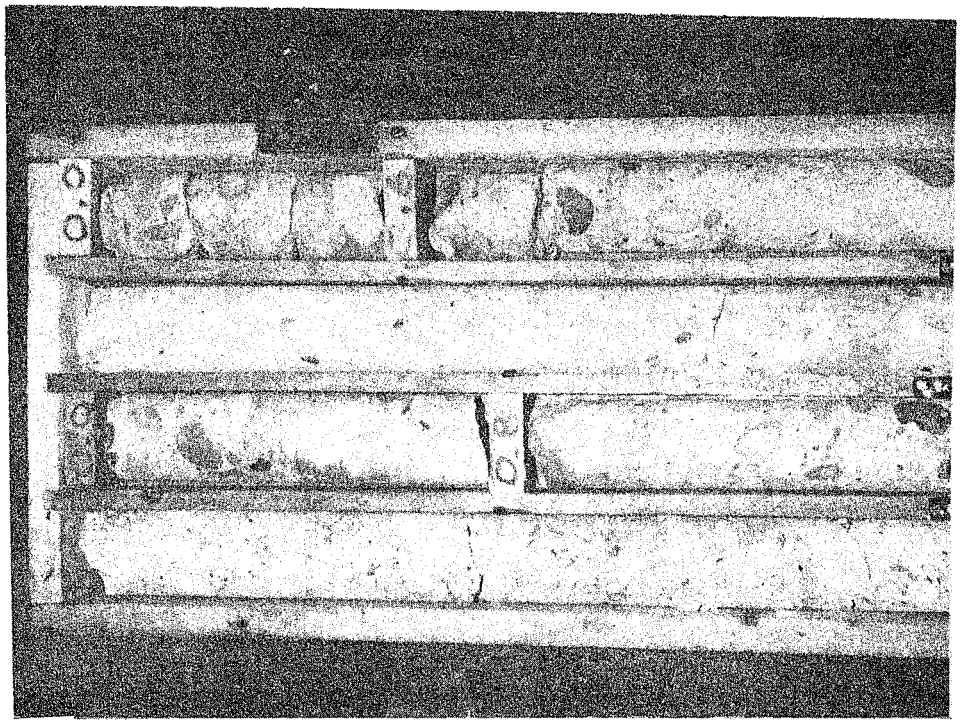


Figure 131. Horizontal cores taken from the land chamber walls, 1973, Emsworth Locks

period. River traffic would be diverted through the river chamber during the land chamber closures.

290. Rehabilitation of the river lock chamber and upper and lower guard walls included: improving the stability of the river and middle walls by installing rock anchors, resurfacing and refacing of river lock chamber walls, encasement of the lower guard wall cells, repair or replacement of upper guard wall cells, repairing the pipe and cable crossover, reconditioning the lower lock gate, replacing river wall filling and emptying valves and operating machinery, installing new piping systems, replacing the hydroelectric plant, modernizing the electrical system, replacing river chamber lock gate operating machinery, construction of new control shelters, removal and replacement of the middle wall operations building, and renovation of the river wall operation building.

291. Rehabilitation of the land lock chamber and upper and lower guide walls included: improving the stability of the land wall by installing rock anchors, refacing and resurfacing of the land lock chamber walls and guide walls, installation of complete hydraulic and electrical systems, adding a new supplemental filling system in the land wall, replacing culvert valves and operating machinery, replacing land chamber miter gate operating machinery, installing wall armor, installing floating mooring bitt, installing tow haulage and retrieval systems, repairing crossover tunnel, constructing new land wall service building, constructing access road, and removing the existing land wall power house.

292. Bids for rehabilitation of Emsworth Locks and Dams were opened in September 1981. The low bid of $24,285,989 was submitted by Morrison-Knudsen Company, Darien, Connecticut (Appendix A). The rehabilitation contract was awarded to Morrison-Knudsen in October 1981. Responsibility for administration of the rehabilitation contract was transferred to the Huntington District in November 1981.

293. Vertical surfaces of the lock walls were resurfaced with both conventional cast-in-place concrete and shotcrete. Resurfacing within the lock chambers extended from 1 ft below lower pool to the top of the walls. The resurfacing outside the chambers extended from the controlling pool line to the top of the walls. A 12-in. thick overlay of reinforced concrete was placed on top of the lock walls. Details of the various types of repairs are shown in Figures 132-135.

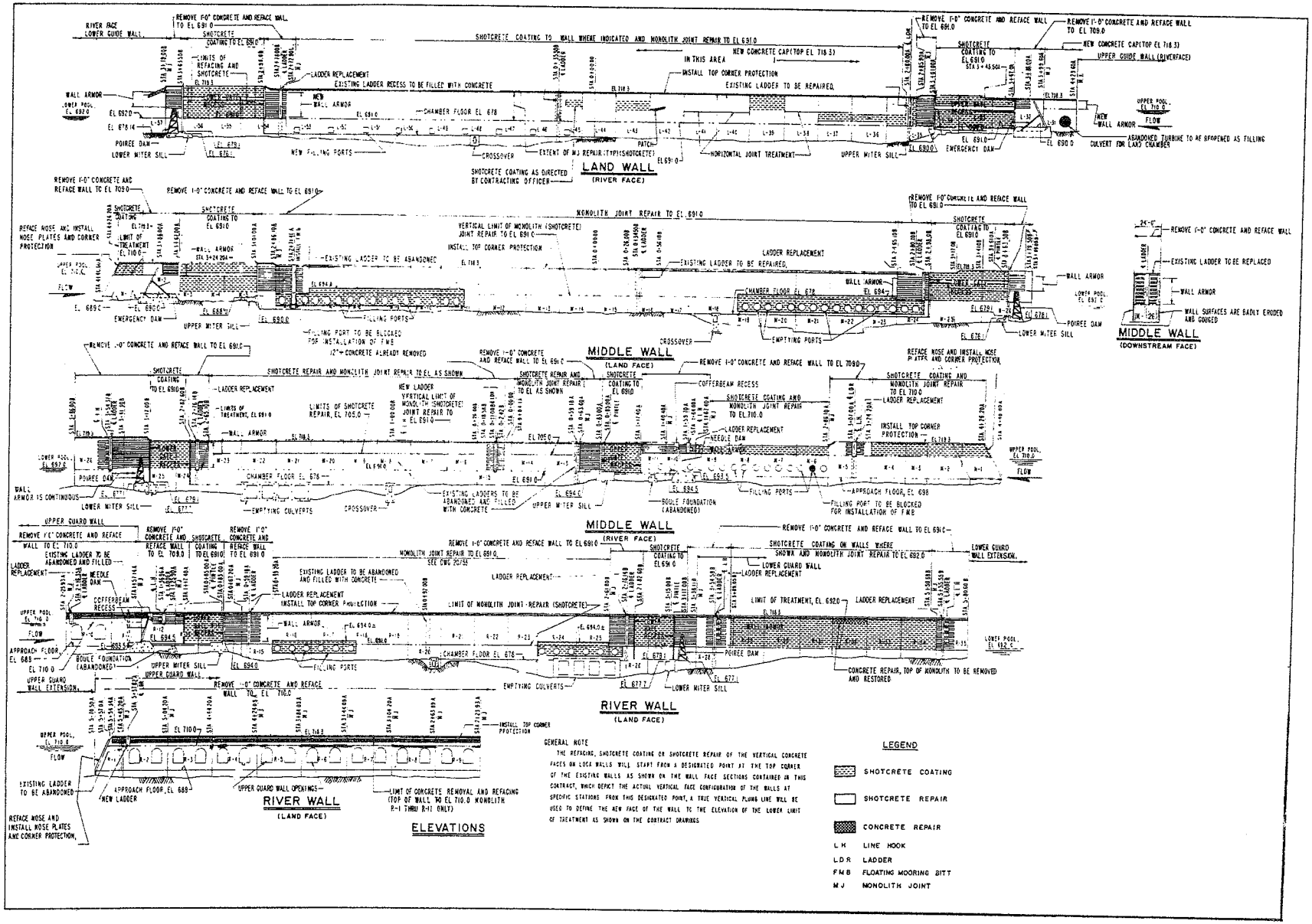


Figure 132. Elevations, lock chamber wall refacing, Emsworth Locks

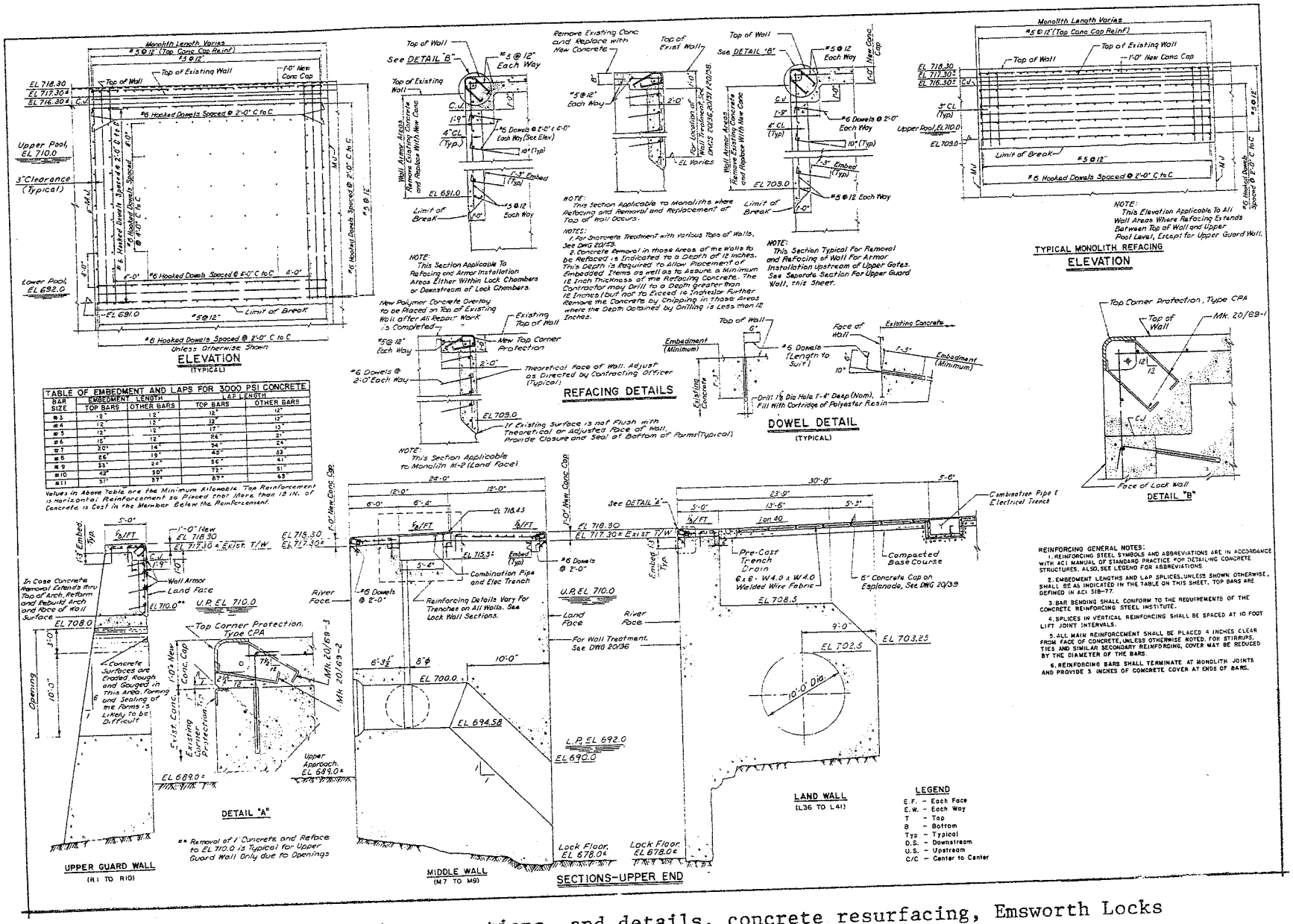


TABLE OF EMBEDMENT AND LAPS FOR 3000 PSI CONCRETE

BAR SIZE	EMBEDMENT LENGTH		LAP LENGTH	
	TOP BARS	OTHER BARS	TOP BARS	OTHER BARS
#3	12"	12"	12"	12"
#4	12"	12"	12"	12"
#5	12"	12"	17"	13"
#6	15"	12"	24"	21"
#7	20"	14"	34"	24"
#8	26"	19"	43"	33"
#9	33"	24"	56"	41"
#10	42"	30"	72"	51"
#11	51"	37"	87"	63"

Values in Above Table are the Minimum Allowable. Top Reinforcement is Horizontal Reinforcement so Placed that More than 12 IN. of Concrete is Cast in the Member Below the Reinforcement.

Figure 133. Elevations, sections, and details, concrete resurfacing, Emsworth Locks

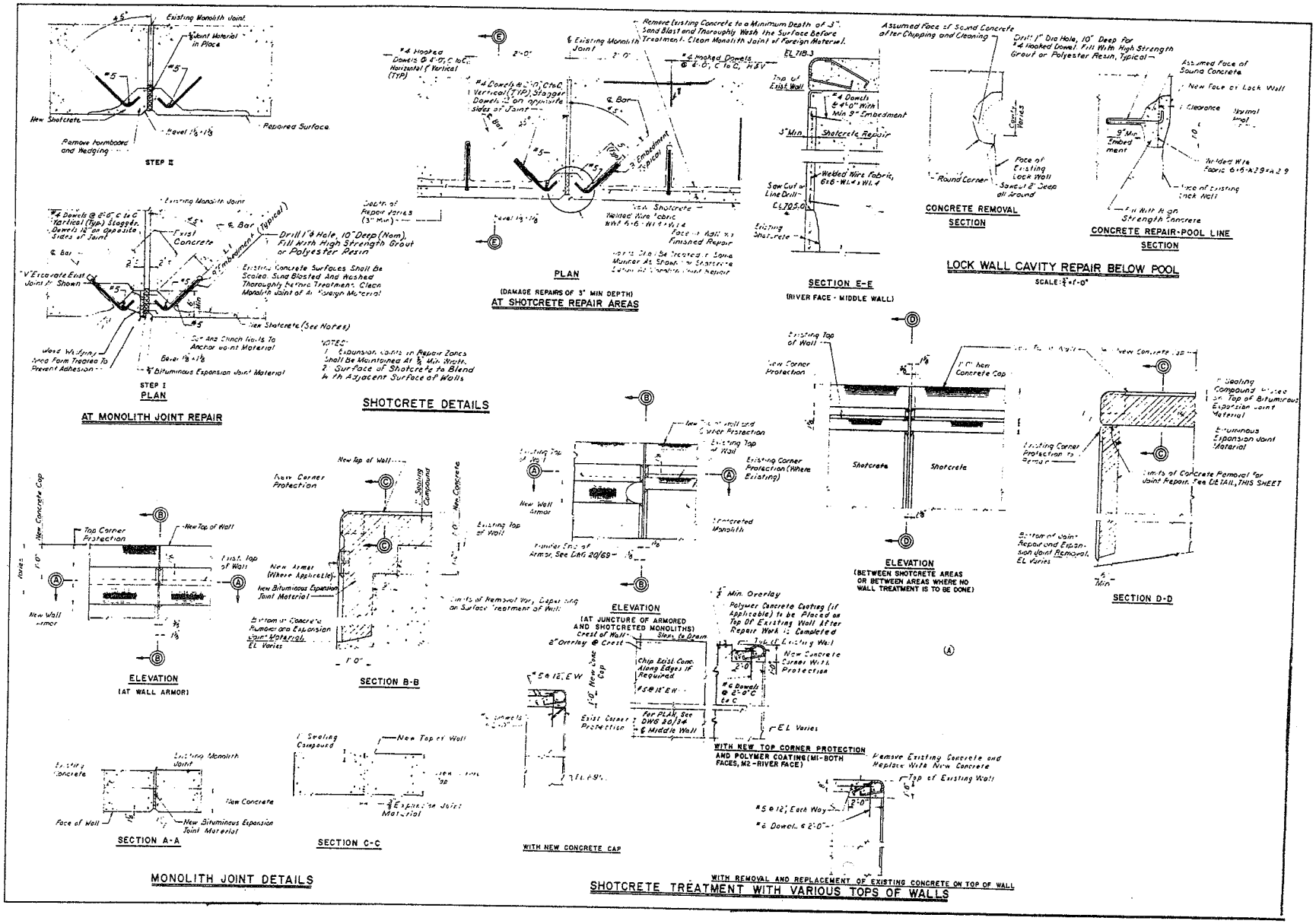


Figure 134. Monolith joint details, Emsworth Locks

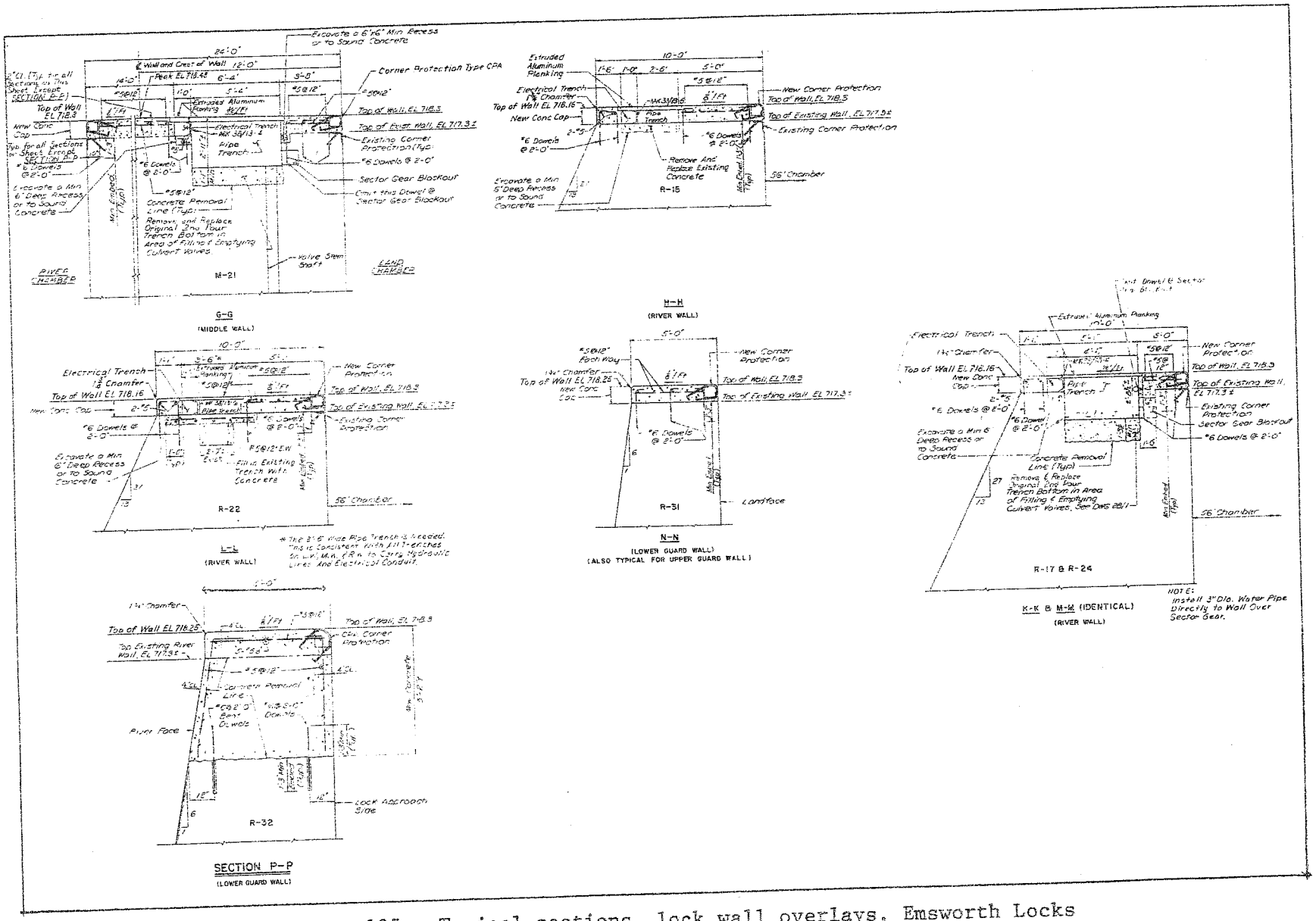


Figure 135. Typical sections, lock wall overlays, Emsworth Locks

294. The sequence of work for resurfacing the lock walls with concrete was as follows:

a. Deteriorated concrete was removed to a depth of 12 in. by controlled blasting. The blasting operation was monitored and recorded to ensure that the resulting vibrations were not of sufficient magnitude to damage the structure. Following removal, concrete surfaces were cleaned using high-pressure water jets.

b. Install dowels to anchor the replacement concrete using polyester resin grout cartridges.

c. Install reinforcing mats on the hooked dowels.

d. Erect form work.

e. Place air-entrained concrete with specified strength of 3,000 psi at 28 days age. Concrete with a maximum water-cement ratio of 0.50 and 1-1/2 in. maximum size coarse aggregate was specified.

f. Remove forms and dry pack all voids and form-work bolt holes.

g. Cure concrete using a membrane curing compound.

295. Areas to be resurfaced with shotcrete were prepared by removing existing concrete or shotcrete to a depth of not less than 3 in. Where areas of previously applied shotcrete were being resurfaced, surface preparation extended from the lower edge of the existing top corner protection down to the point that removal indicated a 3-in.-minimum thickness of sound shotcrete which contained wire-mesh reinforcing. Removal by blasting or high-energy impactors was not permitted. Following removal, the remaining surface was cleaned by wet sandblasting, and welded wire reinforcement was anchored to the wall.

296. The shotcrete mixture was proportioned to obtain a compressive strength of 4,000 psi to 28 days age. A total air content of between 4 and 7 percent, as determined by tests on samples taken as the mixture was placed in the delivery equipment, was specified. Slumps were specified to be within 2 to 4 in. Shotcrete was applied by the wet-mix process. Repairs up to 8-in. thick were successfully shotcreted in a single application using an accelerator in the shotcrete to inhibit sagging. Where repairs were not completed in a single application, underlying layers of shotcrete were continuously moist cured for 7 days or until placement of subsequent layers. The finished surface was cured with membrane curing compound.

297. A shotcrete coating was applied to selected areas of the lock walls (Figure 132). This coating consisted of layers of shotcrete built up as

required to provide a minimum shotcrete thickness of 3/4 in. over the existing surface. Where the existing surface was spalled or deteriorated, the shotcrete applied after surface preparation was of sufficient thickness to restore the surface to the original location plus 3/4 in. The shotcrete coating did not contain reinforcement.

298. Monolith joint repairs consisted of preparing the joint, installing dowels and reinforcement, and shotcreting the edges of the monolith as shown in Figure 134. In areas which were resurfaced with shotcrete or received a shotcrete coating, the monolith joint repair extended 8 in. on both sides of the joint. In other areas, spalled areas along the joint received a shotcrete coating to the extent required for blending into adjacent concrete surfaces.

299. The top surfaces of the lock chamber walls received a 12-in.-thick overlay of reinforced concrete (Figure 135). The horizontal surfaces were cleaned with a high-pressure water jet, and dowels were installed to anchor the replacement concrete. In addition, application of an epoxy bonding compound to the cleaned surfaces immediately prior to concrete placement was specified.

300. The top surfaces of monoliths M-1 and M-2 received a polymer mortar overlay. A two-component acrylic copolymer mortar designed, manufactured, and marketed as a patching and overlay material for concrete surfaces subject to weathering was specified. Deteriorated concrete was removed by chipping with hand tools or a pneumatic hammer not heavier than the nominal 30-lb class. After concrete removal, the surface was prepared and the mortar was mixed, placed, and finished in accordance with the mortar manufacturer's recommendations. Wet curing for 48 hr was specified.

301. A condition survey of the Emsworth project was conducted in December 1985 as part of a study to project the condition of the project in the year 2010 (Stowe 1986). That portion of the report relating to performance of the rehabilitation accomplished during the period 1982-85 is summarized in the following discussion.

302. The concrete on the top surface of the land wall was in good condition although there were from three to six fine transverse cracks in each monolith. Lock personnel indicated that most of these cracks formed shortly after the concrete was placed. The shotcrete coating applied to portions of six chamber monoliths of the land wall was in generally poor condition. About

80 percent of the shotcrete coating on monoliths L-37 and L-38 between elevations 692 and 703 was missing to depths of up to 2 in. The coating on monolith L-39 contained a reflective vertical crack which was also present in the concrete below the lower limit of the shotcrete coating. The monolith joint and ladder-way repairs were in good condition. The shotcrete coating applied in the gate recesses was in poor condition with numerous horizontal and diagonal cracks (Figure 136). The majority of these cracks, which ranged in width

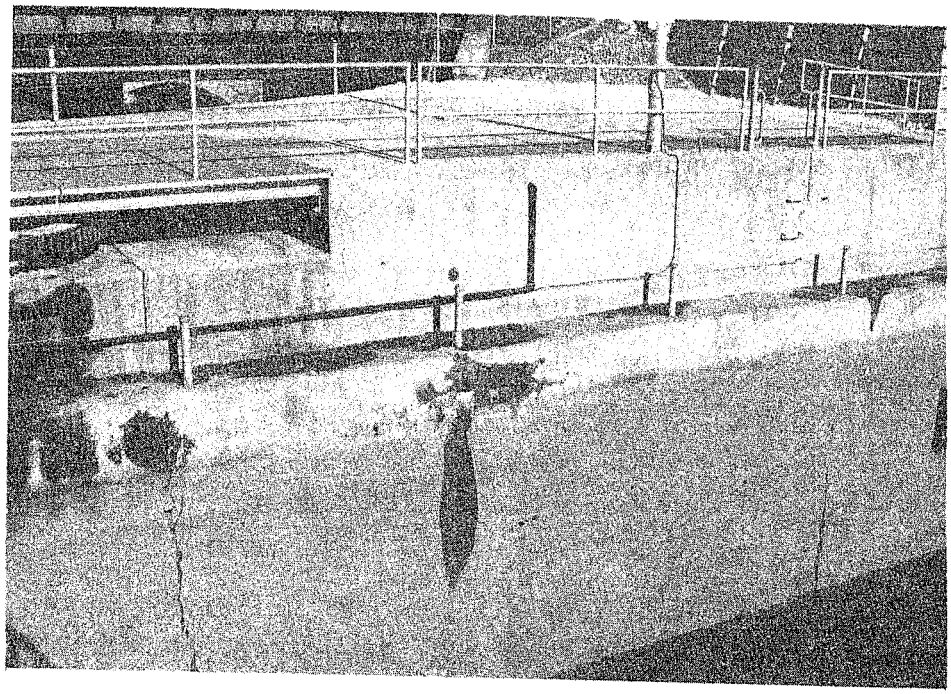


Figure 136. Cracking in shotcrete coating, downstream gate recess, land wall, December 1985, Emsworth Locks

from fine to wide, were located above upper pool elevation. According to lock personnel, these cracks occurred almost immediately after shotcreting, perhaps because of the prevailing hot, dry wind at the time of placement. Efflorescence was associated with a number of these cracks. A few cracks were dampened by water, and in several places water seepage through cracks in the coating covered large areas of the coating. Small areas of the shotcrete coating were debonded from the underlying concrete.

303. The concrete on the top surface of the middle wall was in good condition with the exception of two monoliths which exhibited three to six fine transverse cracks. These cracks extend through the 12-in. concrete overlay as evidenced by their presence on both faces of the wall for the full depth of the overlay. The polymer mortar on the top surface of monoliths M-1

and M-2 was in poor condition with extensive pattern cracking (Figure 137). These cracks allow water ponding on top of the monoliths to infiltrate the underlying concrete as evidenced by water seepage on vertical wall surfaces beneath the corner protection armor. The ladder ways and monolith joints repaired with concrete and shotcrete, respectively, were in good condition.

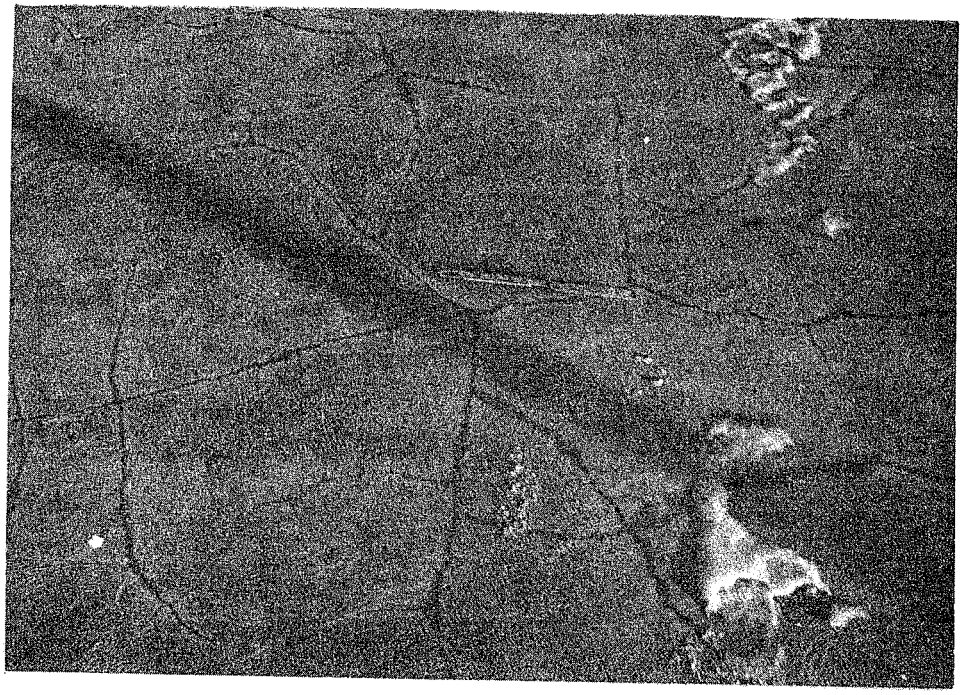
304. The concrete resurfacing on monoliths M-25 and M-26 was also in good condition. The shotcrete coating applied to the upstream gate recess in the land face of the middle wall was in fair condition. The coating contained five diagonal wide cracks and two vertical cracks with dark staining and light efflorescence associated with the cracking. The coating in this recess was in somewhat better condition than the coating in the opposite recess in the land wall. Since the land face of the middle wall is shaded from sunlight, it probably undergoes fewer cycles of freezing and thawing than the opposite wall which is exposed to sunlight. The shotcrete coating in the downstream recess was in fair condition above high pool elevation and in good condition between low and high pool elevations. Above upper pool elevation it contained localized areas of pattern cracking and a few isolated horizontal cracks containing small spalls with efflorescence. Again the coating was in better condition than the opposite recess in the land wall.

305. The condition of repairs to the river face of the middle wall are summarized as follows:

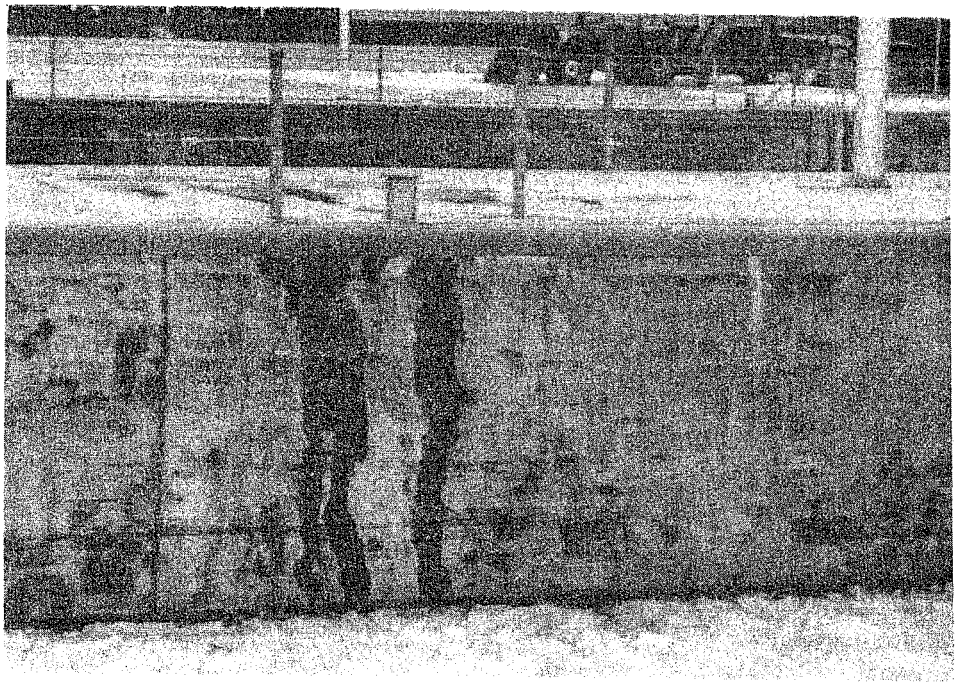
Monolith No. or Element	Shotcrete Coating	Shotcrete Resurfacing	Concrete Resurfacing	Remarks
M-1 thru M-4	Poor			Numerous fine and wide vertical cracks (at waterline up to 13 per monolith), water seepage, soak staining
M-5			Good	Two wide diagonal cracks
M-6 thru M-9	Fair			Numerous fine and wide cracks (at waterline up to 15 per monolith)
M-10 and 1/2 M-11*			Good	

(Continued)

* About one-half width of monolith.



a. Pattern cracking, top surface



b. Seepage on chamber face as a result of water ponding on top surface

Figure 137. Condition of polymer mortar overlay, monolith M-2, December 1985, Emsworth Locks

Monolith No. or Element	Shotcrete Coating	Shotcrete Resurfacing	Concrete Resurfacing	Remarks
1/2 M-11 and 1/2 M-12			Good	GATE RECESS - appears to be concrete; shotcrete called for. Two fine vertical cracks
1/2 M-12			Good	
M-13 and M-14		Fair		Fine and medium diagonal cracks
M-15			Good	Three wide vertical cracks
M-16 thru M-23		Fair		Fine to wide vertical and horizontal cracks
1/2 M-24			Good	
1/2 M-24 and 1/2 M-25	Good			GATE RECESS - minor fine vertical cracking and minor leaching
1/2 M-25 and M-26			Good	
Ladder ways			Good	
Monolith joints		Good		

The deterioration of the shotcrete coating on monoliths M-1 through M-4 and M-6 through M-9 is thought to be the results of freezing and thawing action, and it will likely continue to the point where large areas will become debonded, as in monoliths L-36 and L-37 of the river face of the land wall.

306. The concrete on the top surface of the river wall was in good condition although transverse cracking similar to that on the other walls was observed. What appeared to be concrete resurfacing in the upstream gate recess was in good condition although one wide vertical crack was present beneath the sector arm. The shotcrete coating applied to the face of the downstream gate recess in 1982 was in good condition. Several fine vertical cracks were present in the shotcrete above upper pool elevation. The shotcrete coating applied to the river face of the river wall in 1985 was apparently in good condition. The vertical section of the wall could not be fully viewed; however the sloped section could be observed and no deficiencies were noted.

307. No deficiencies were noted in the concrete overlay on the upper guard wall except for some fine transverse cracks. Each monolith contained three to six cracks that were somewhat evenly spaced. All cracks extend the full depth of the overlay as viewed on the river face. The shotcrete coating on the river face of the wall was in good condition. The armored concrete on the land face was in good condition.

308. The concrete on top of the lower guard wall was in good condition although each monolith exhibited one fine transverse crack near its midpoint. The shotcrete coating on both faces of the wall was in good condition. The shotcrete coating on the landside had reddish brown stains as a result of barges' rubbing against high spots in the shotcrete, but there were no appreciable signs of abrasion.

Lockport Lock

309. Lockport Lock is located at river mile 291 on the Illinois Waterway immediately west of the city of Lockport. The lock is 600 ft long by 110 ft wide and has a lift of 39 ft (Figure 138). The lock walls and sills are of concrete masonry. Two submersible vertical lift gates, a guard gate and a service gate, are provided at the upper end of the lock. The lower gates are of miter type. The filling and emptying system is the wall-port type. The lock was completed in 1933 at a total cost of $2,153,867. Since the original construction, the following major rehabilitation was performed:

a. Floating mooring bitts were installed in 1961.

b. The top of the lock walls was resurfaced with 4 in. of concrete in 1966.

c. Culvert valves were replaced and hydraulically operated valve machinery was installed in 1966-1968.

d. Upper lock gates and lifting chains were replaced in 1968-1969.

e. Mooring Pier No. 1 was repaired in 1974.

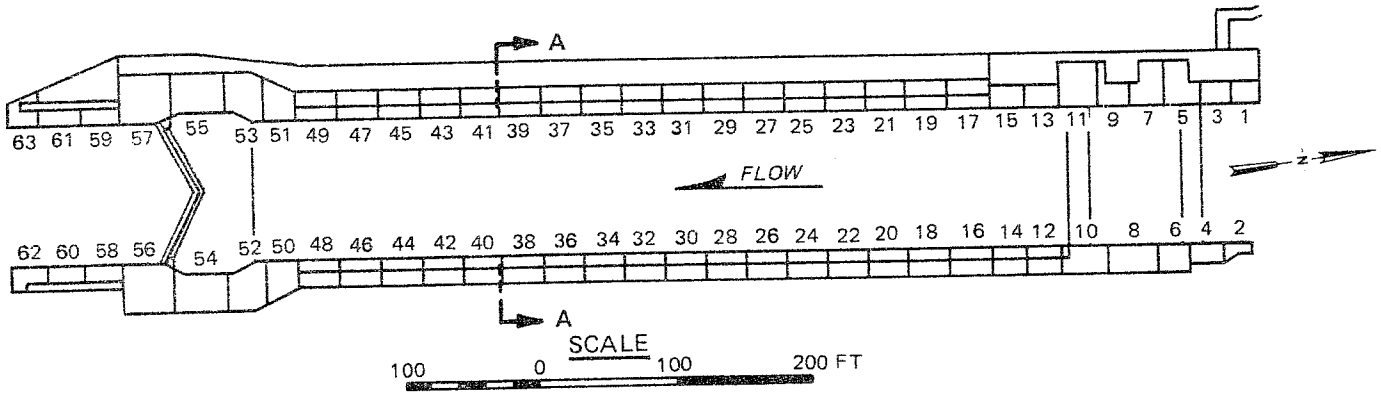
f. Electrical system was replaced in 1976.

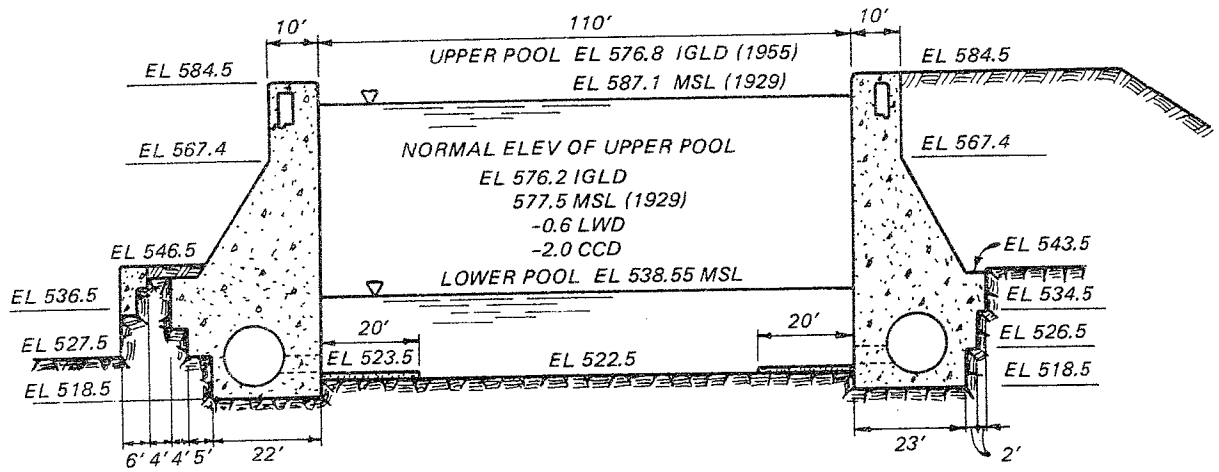
310. In 1978, Waterways Experiment Station (WES) was requested to ascertain the extent of concrete deterioration and determine selected physical properties of concrete and bedrock at Lockport Lock. Results of a crack survey, core drilling, field and laboratory testing of concrete, and laboratory testing of foundation rock as reported by Stowe et al. (1980) are summarized below:

a. New air-entrained concrete placed during resurfacing of the tops of the lock walls was in good condition.

b. The old nonair-entrained concrete was lightly to severely deteriorated primarily as a result of freezing and thawing.

c. Average depth of damaged concrete ranged from 0.2 ft in the lock chamber walls to 1.3 ft in the lower gate bay.





SECTION A-A

Figure 138. Plan and typical section, Lockport Lock

d. Severe damage existed at monolith joints, especially on the lockside of the river wall.

e. Beyond the damaged concrete zones, the concrete was strong with an average compressive strength of more than 6,000 psi.

f. A significant crack was located in the land lock wall.

g. Physical properties of the foundation rock previously determined were verified.

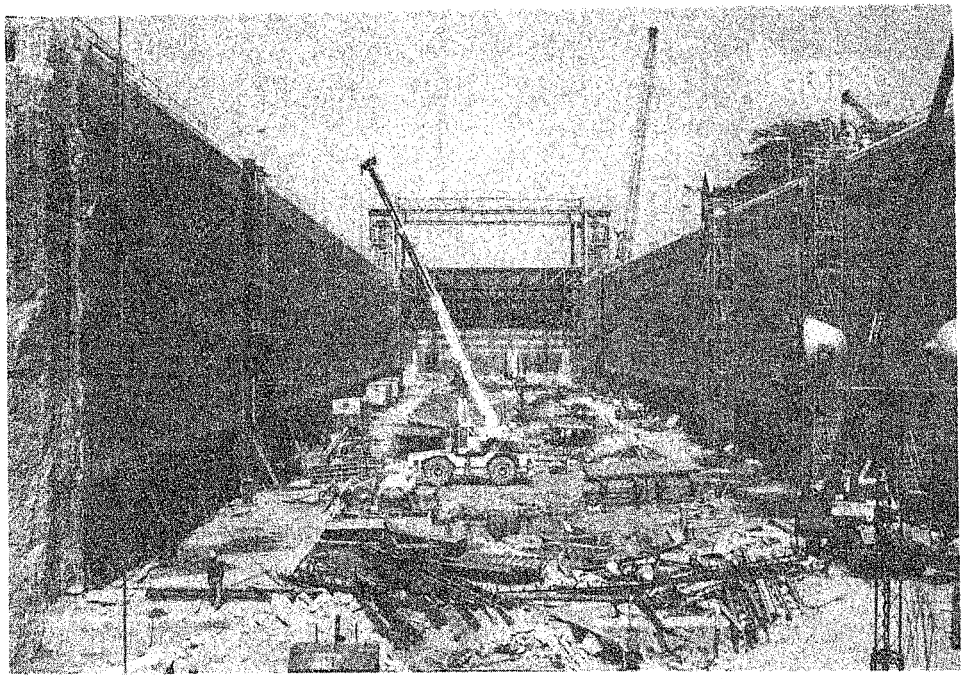
311. The basis for design of the Stage I Rehabilitation of Lockport Lock is contained in Design Memorandum No. 1 (Rock Island District 1982). This report contains an evaluation of the existing condition of each feature of the lock and the necessity for performing rehabilitation work to extend the useful life of the major structures for 50 years. There was concern about the stability of the lock walls, and extensive stability evaluations were made. Results indicated that the walls would not meet current stability criteria if evaluated conservatively for full uplift and at-rest soil pressures. Bedrock boundary conditions at the riverward toe of the river wall made the analysis complicated and somewhat uncertain. Earlier finite element analyses showed that the walls were stable, and indeed 50 years of operation bore this analyses out. However, stabilization was still a requirement at the time Stage I was started. Further analyses were made to assure the safety of dewatering for Stage I, which would occur before the stabilization could be installed. It was decided to instrument the lock walls to detect any movement. Minor movements on the order of a few thousandths of a foot were detected randomly, some in the direction opposite to the major forces. It was concluded that either the movement was too small to measure with ordinary surveying methods or temperature changes probably were causing some unaccounted for movement. Further analyses showed that if fairly small movements occurred inward during dewatering, the backfill pressures would be reduced considerably. The wall would thereafter be safe. It was considered safe to proceed with Stage I. Subsequently, after further analyses, the walls were declared stable, and plans to stabilize them were dropped. The upper sill was considered to need additional stabilization, and rock anchors were designed and installed for this purpose.

312. Based on the condition survey and structural analysis, plans and specifications were prepared for a multistage rehabilitation of Lockport Lock. J. A. Jones Construction Company of North Carolina was the low bidder for Stage I rehabilitation of the lock. Bids ranged from $8.1 to $10.4 million as

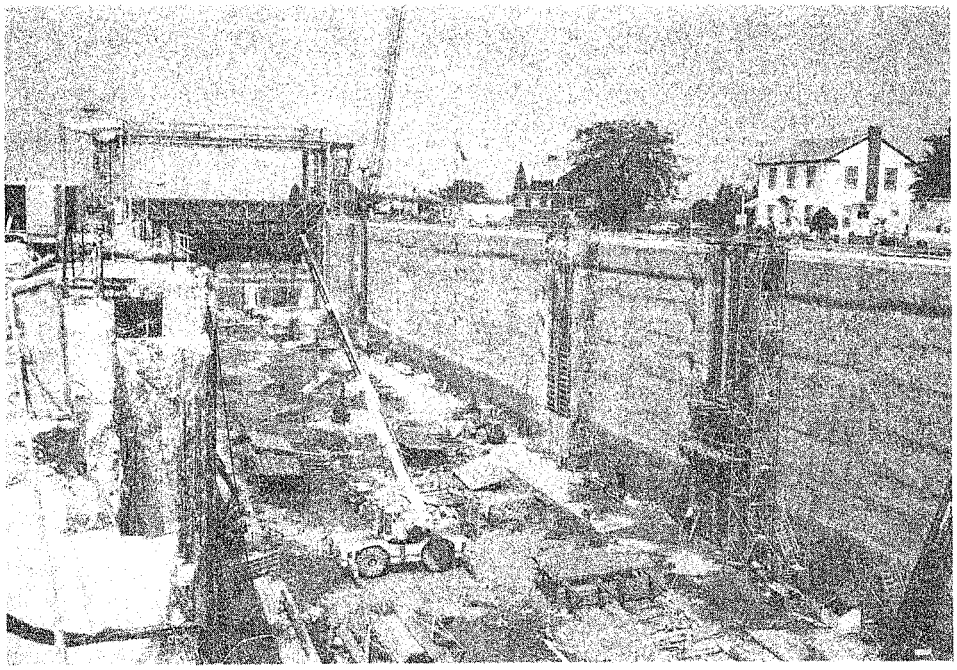
compared to the Government estimate of $8.0 million. An abstract of the bids is shown in Appendix A. The major work items under this contract were complete replacement of the lower miter gates and machinery, stabilization of the upper service gate sill, modifications to the upper lift gates, modification of the electrical distribution system, and miscellaneous concrete resurfacing (Figure 139).

313. The major effort in the rehabilitation of Lockport Lock involved the lower miter gates. The lock was closed to navigation on 5 July 1984, and the first operation prior to placement of the dewatering bulkheads was removal of the existing 315-ton, 60-ft-high miter gate leaves. This operation was accomplished by use of a barge-mounted derrick crane before the lock was dewatered. The miter gates were replaced with a modern all-welded steel design, which required rebuilding of the sill and the embedded (quoin) anchorage. The new sill (Figure 140) required concrete and rock removal, prestressed anchors, and installation of new embedded metals for the new seals. Water leakage through the sill required the installation of a well point system to intercept all water coming from joints and planes in the bedrock. The new quoin anchorage required removal of concrete around and including the existing quoin (Figure 141). The contract required removal of this concrete by nonblasting techniques because of the sensitivity of the area. The contractor chose to drill lines of small diameter holes along the top of the lock walls parallel to the lock chamber and to fill alternating holes in the row nearest the chamber with S-mite, an expansive grout. This attempt to presplit the concrete along the drill line resulted in cracks in the concrete which were very erratic in both direction and extent. Cracking behind the quoin anchorage area was of particular concern.

314. In order to complete the necessary concrete removal, the remaining holes along the first drill line were loaded with detonating cord and the fractured concrete removed by blasting (Figure 142). A similar procedure was used for succeeding lines of drill holes. Extensive concrete removal by hand using jackhammers was still required to complete the removal operation. The crack behind the quoin anchorage area became a critical path item since the lock could not be reopened until this problem was resolved. The question was whether to remove the loose piece of concrete entirely or to anchor it in place. Since the cracked section of concrete is loaded in compression with the lock at upper pool, it was concluded that it could be posttensioned to the

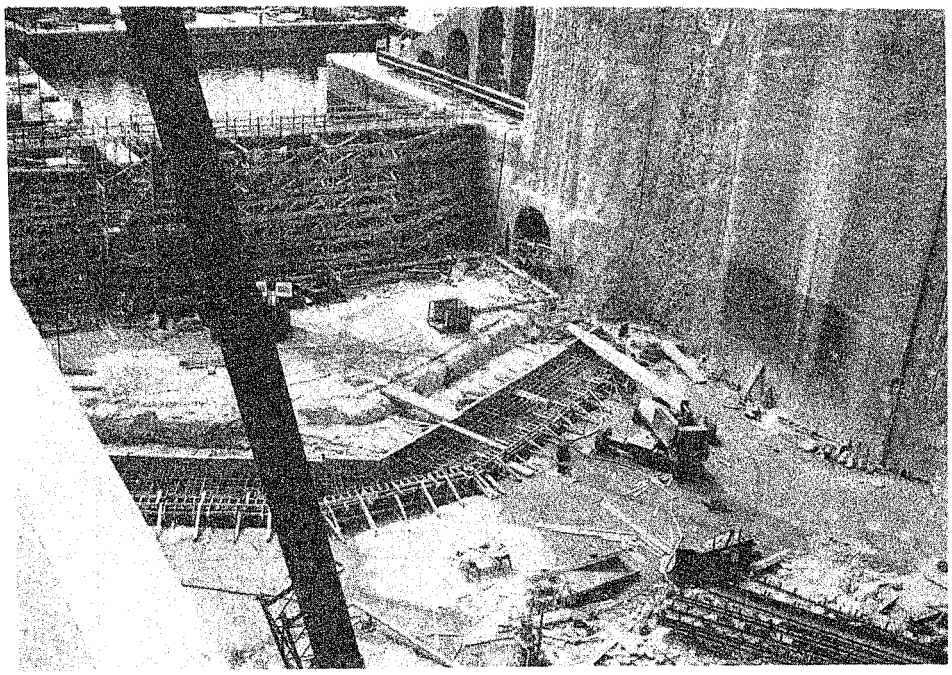


a. Overall view of lock chamber

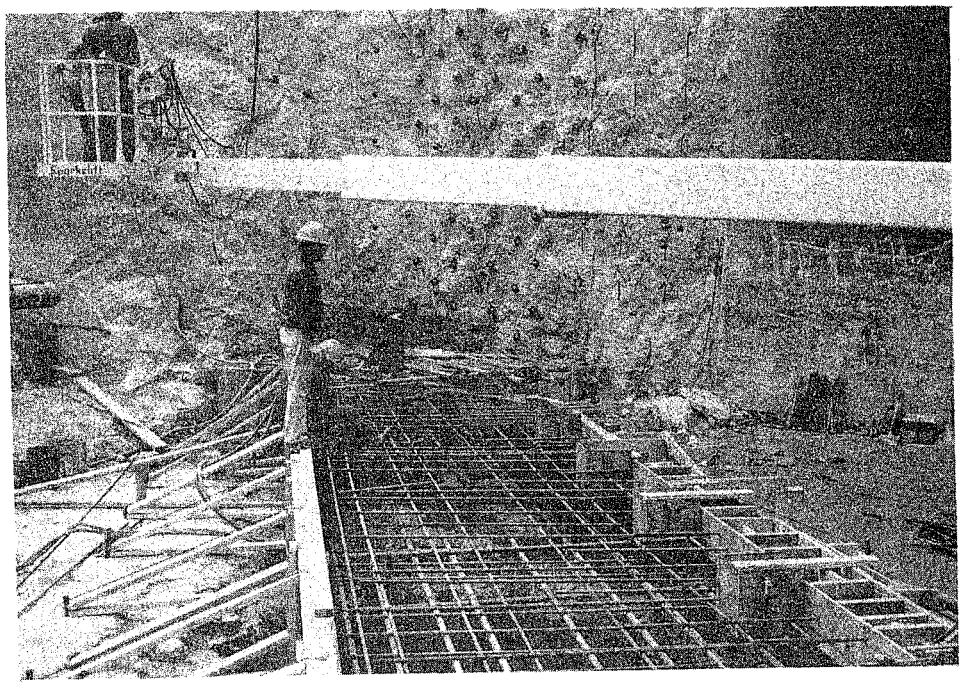


b. General view of landside lock wall

Figure 139. Stage I rehabilitation, Lockport Lock

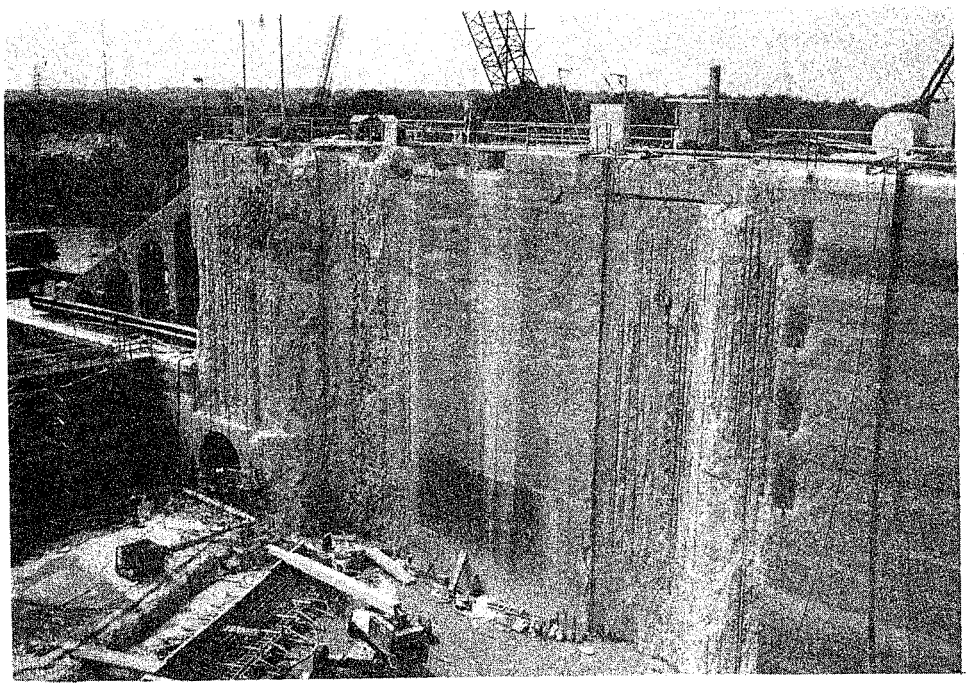


a. Overall view

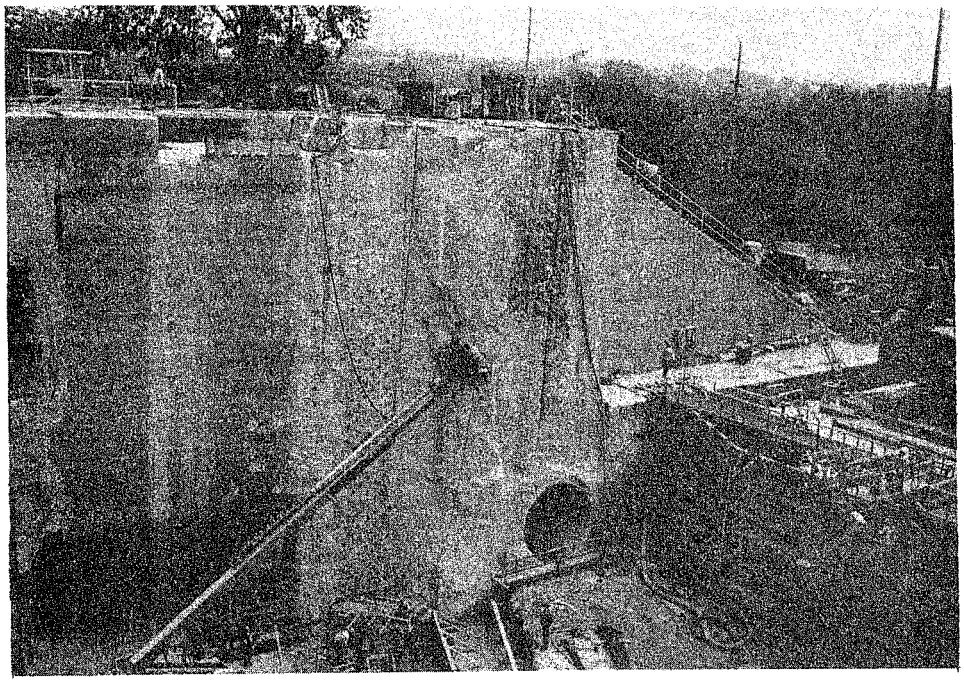


b. Closeup view

Figure 140. Miter gate sill and anchorage, modifications, Lockport Lock



a. Riverside wall



b. Landside wall

Figure 141. Lock chamber modifications to accommodate new miter gates, Lockport Lock

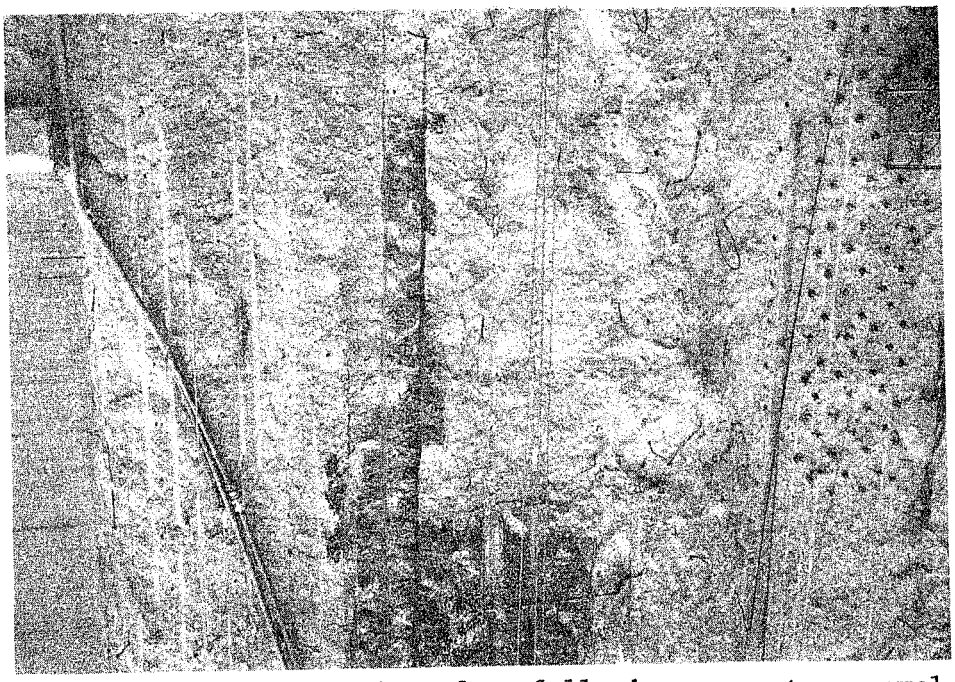


Figure 142. Typical surface following concrete removal, Lockport Lock

main monolith mass with no loss in structural integrity. Therefore, a number of anchors were installed to strengthen the concrete sections near the miter gate anchorages (Figure 143). Following installation of dowels, gate anchors, and conventional reinforcing (Figure 144), these areas were formed and a concrete overlay was placed over the entire area to seal all internal work. A concrete mixture (Type C) proportioned for a slump of 1 to 4 in., an air content of 6 ± 1-1/2 percent, and 4,000-psi compressive strength at 28 days was used in all concrete work with the exception of the upper service gate sill. Concrete mixture proportions for a 1-cu yd batch were as follows:

Type I portland cement, lb	517
Natural sand, lb	1,390
Limestone coarse aggregate, lb	1,750
Water, gal	30
Water reducer, oz	15.5
Air-entraining admixture, oz	4.0

The concrete was mixed in transit mixers and transported to the project site by Meyer Material Company, Des Plaines, Illinois. This major repair is performing well with no adverse operational or visual structural problems (Rock Island District 1985).

315. After completion of the quoin anchorage and new sill, the new

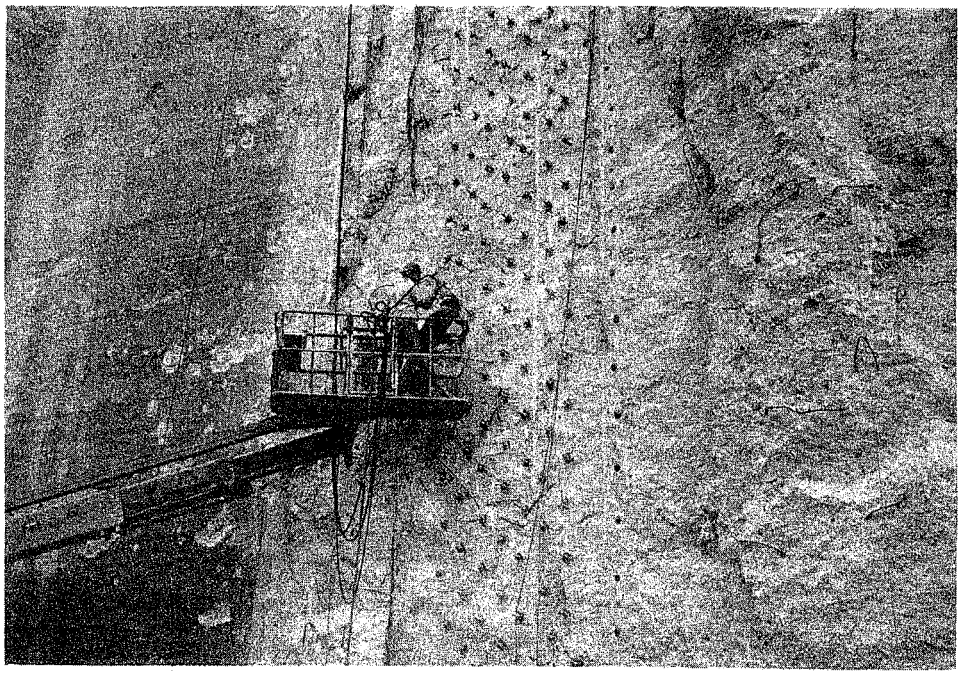
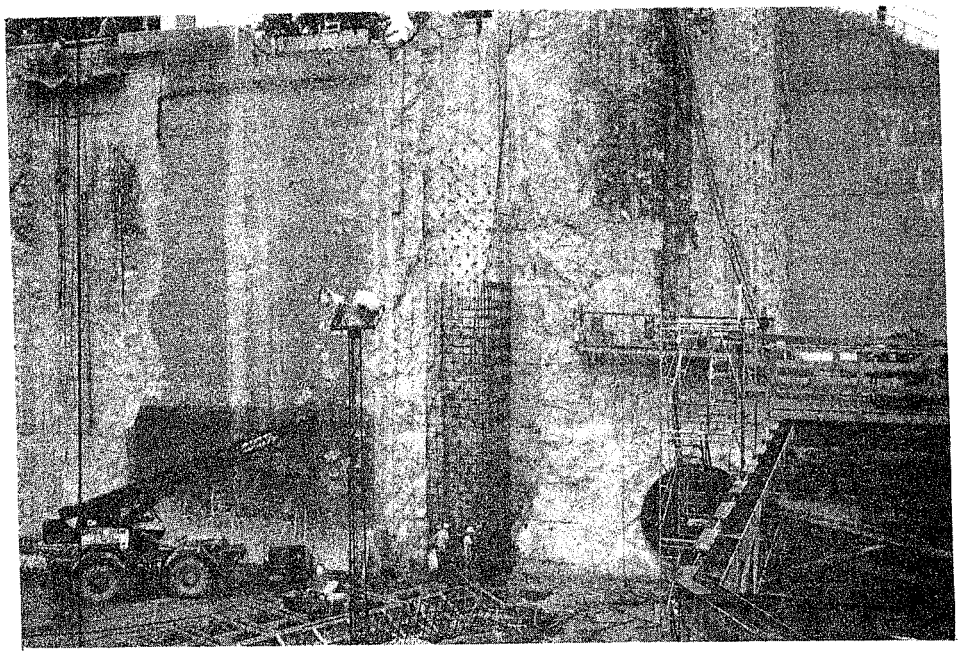


Figure 143. Rock anchors used to strengthen concrete sections near the miter gate anchorages, Lockport Lock

miter gates were set in place. The new gate design resulted in a reduction of the total weight of each gate leaf from 315 to 250 tons. The same barge-mounted derrick that removed the old gates was used to place the new gate leafs. The gates were set on the pintle and attached to the anchorage links on top of the gate. The miter blocks and quoin blocks were adjusted to provide as tight a seal as possible during normal gate operation. After adjustments were made, the space between each block and its respective backing plate was filled with babbitt metal. Epoxy material was originally called for as the fill material, but the cool weather at the time required a long set time for epoxy. Time constraints did not permit the long set time, so hot babbitt was substituted for the epoxy.

316. The upper service gate sill is a gravity structure founded on rock. Early design studies indicated that the sill did not meet current stability criteria. In addition, the underlying bedrock was found to be fractured and jointed. The design required a foundation grouting program to help consolidate the underlying bedrock. This work was accomplished by drilling holes down through the sill and pumping grout into the bedrock. The stabilization program consisted of installing 23 stranded anchors across the 110-ft sill. Prior to drilling holes for the anchors, a slot was cut in the top of



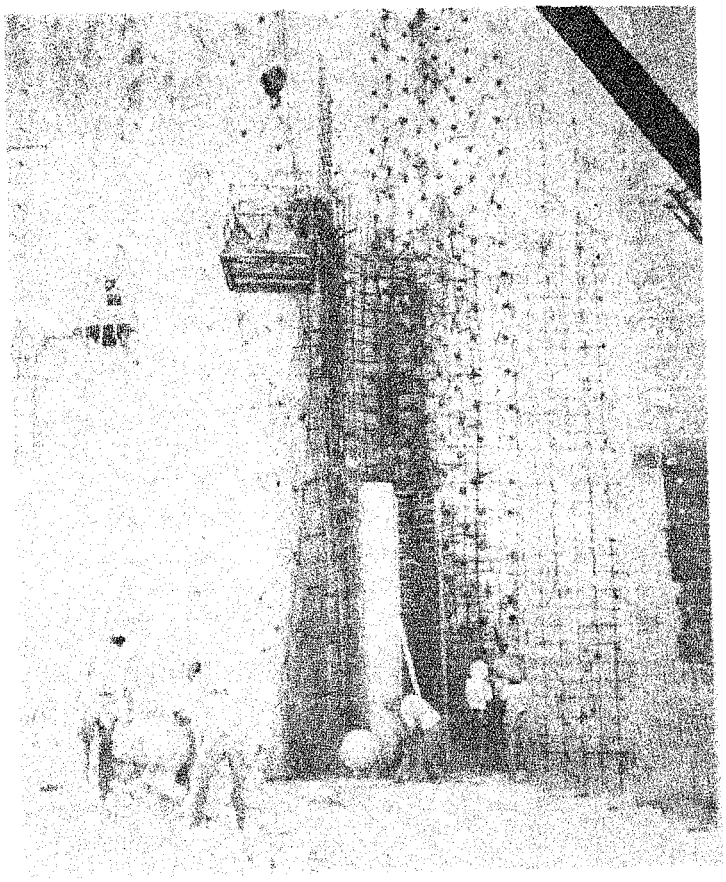


Figure 144. Reconstruction of quoin area and miter gate sill, Lockport Lock

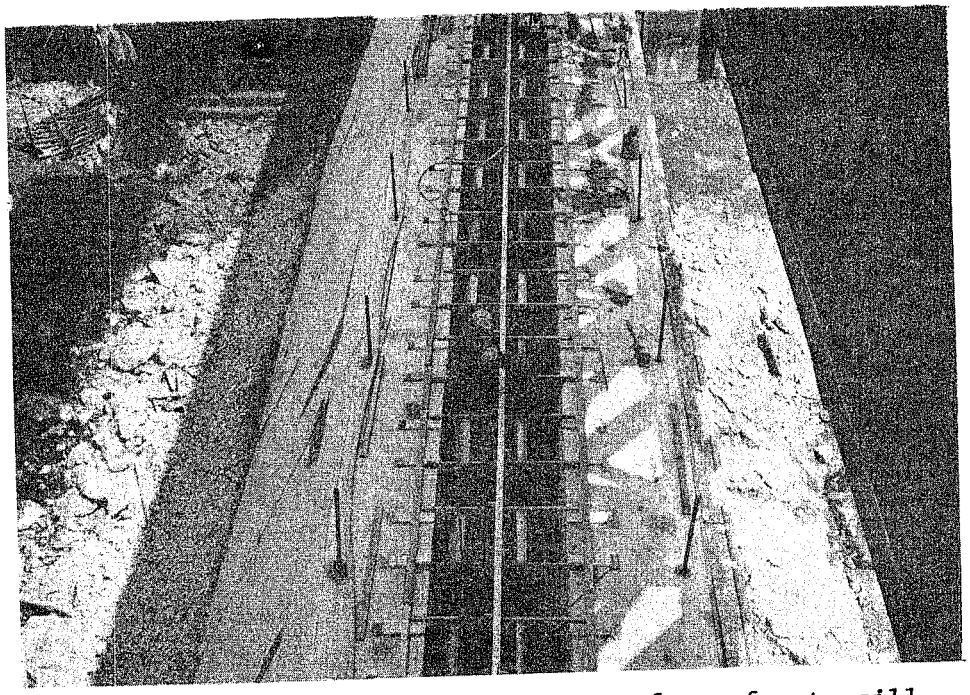
the sill. After reinforcement was installed, blockouts for anchor recesses were formed, and concrete was placed to original grade (Figure 145). A concrete mixture (Type A) proportioned for a slump of 1 to 4 in., an air content of 6 ± 1-1/2 percent, and 4,000-psi compressive strength at 7 days age was used in gate stabilization work. Concrete mixture proportions for a 1-cu yd batch were as follows:

Type I portland cement, lb	611
Natural sand, lb	1,365
Limestone coarse aggregate, lb	1,775
Water, gal	32
Water reducer, oz	18.3
Air-entraining admixture, oz	4.5

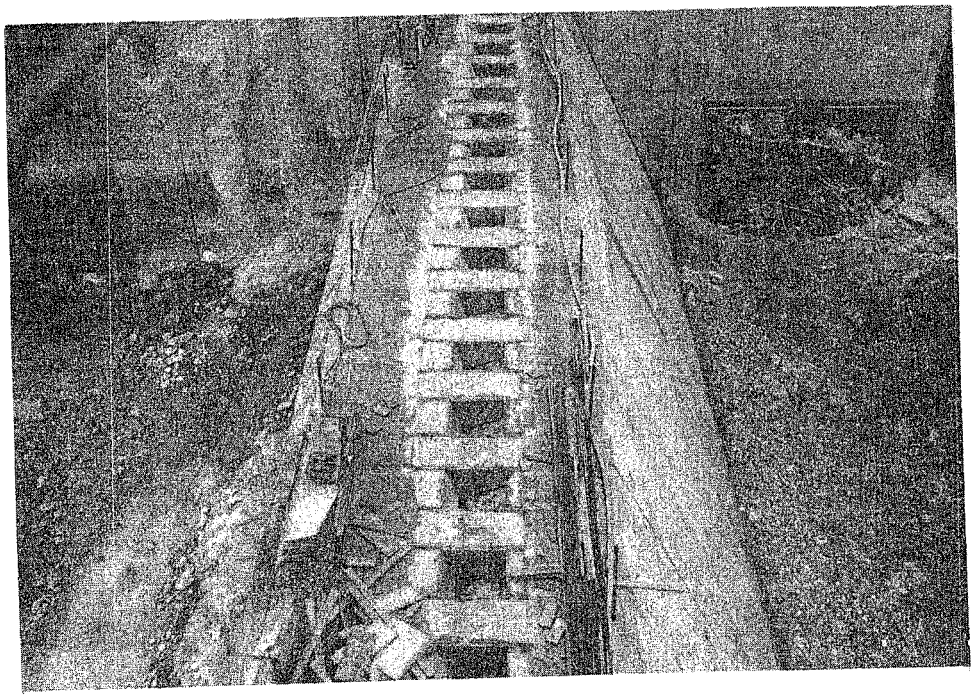
317. After anchor holes were drilled in each recess, the anchors were installed and the embedment lengths grouted. The stranded anchors were manufactured by Dywidag Systems International, Incorporated, Lemont, Illinois. Each anchor consisted of sixteen 1/2-in. strands, 74 ft long, enclosed in a double corrosion protection system (Figure 146), and seated at 455 kips. The design working load of the anchors was 325 kips. Each anchor was lift-off tested immediately after seating. The maximum loss shown by the lift-off test was 6 percent. Twenty-four-hour lift-off tests were conducted on three of the anchors. The maximum loss for the 24-hr lift-off was 7.5 percent. Upon completion of stressing, the remainder of each hole was grouted, and the recesses were filled with concrete. The stranded anchors enabled the sill to meet current stability criteria and ensure the structural integrity of the upper part of the lock.

318. The present lifting system for the upper lift gates at Lockport does not allow operation of the gates under a differential head. Emergency closure was deemed necessary to prevent loss of upper pool during a lower miter gate failure from barge impact. The gates were modified as part of Stage I by enlarging the gate recesses (Figure 147) and replacing the friction pads on the gates with rollers. The future installation of new gate machinery (Stage III) will result in the capability of operating the gates under differential head.

319. The present electrical distribution system to the riverside of the lock extends across the existing lift gate towers. The replacement of lift

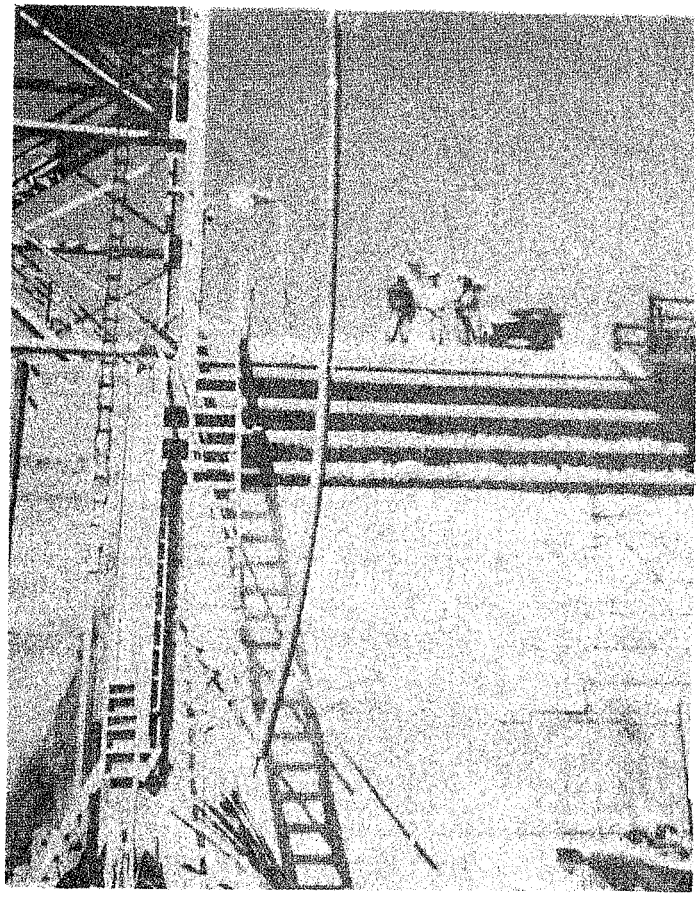


a. Anchor recesses formed in top surface of gate sill

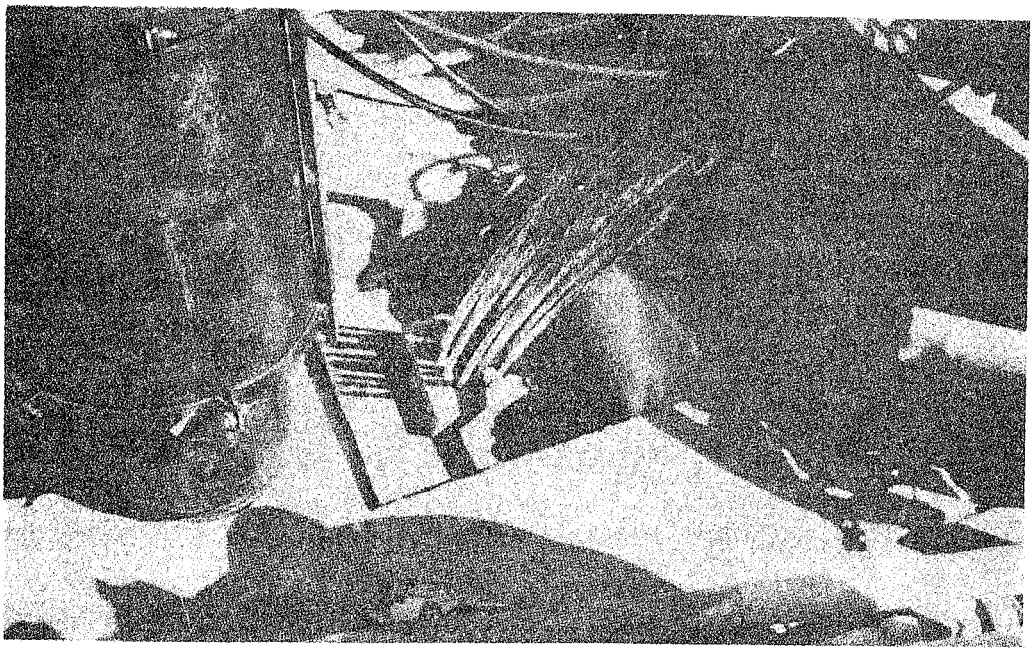


b. Completed anchor recesses

Figure 145. Modification of upper service gate sill prior to installation of rock anchors, Lockport Lock

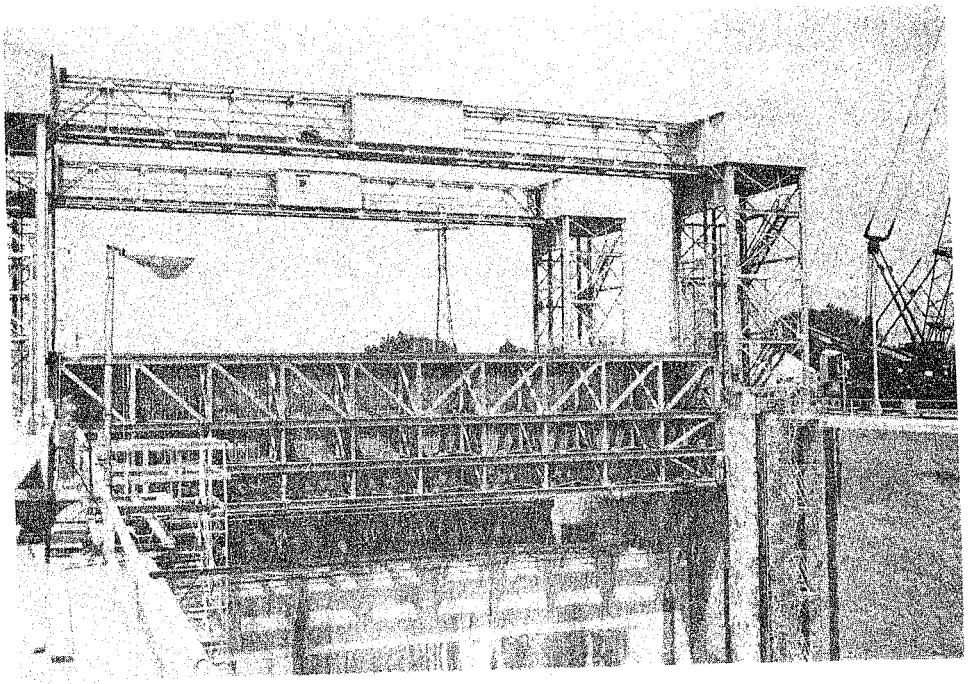


a. Posttensioned anchor being lowered by a crane

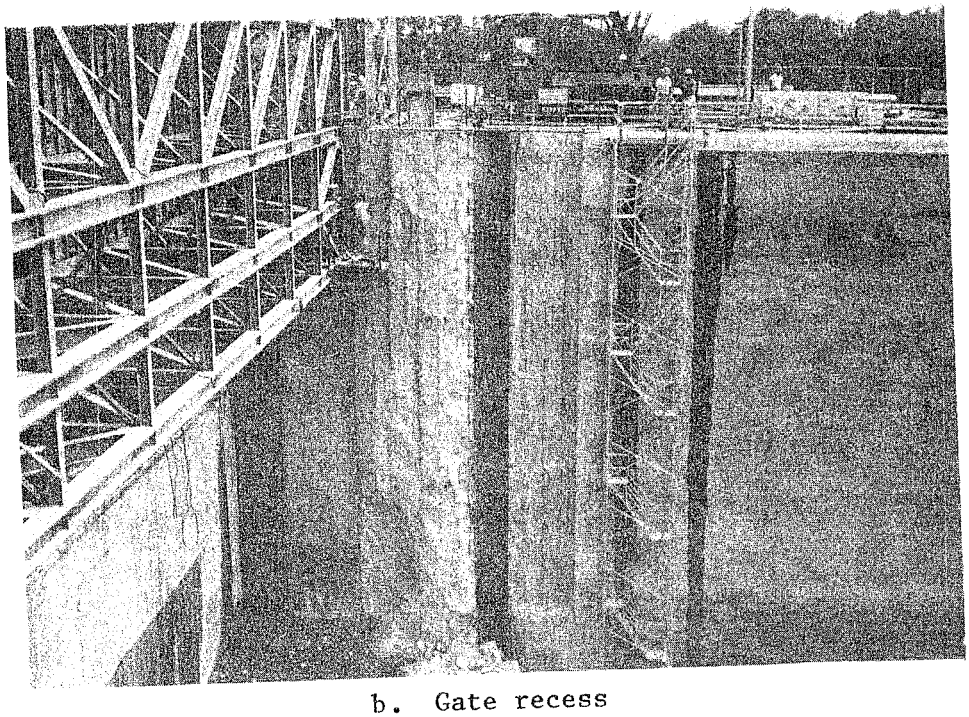


b. Stranded anchor in place prior to posttensioning

Figure 146. Anchor installation, upper service gate sill, Lockport Lock



a. Overall view



b. Gate recess

Figure 147. Upper service gate modifications, Lockport Lock

gate machinery under Stage III will result in complete removal of the machinery towers. The electrical system to the riverside of the lock was relocated in Stage I to a cable trench in the bottom of the lock. The relocation could only be done while the lock was dewatered.

320. The condition survey of the lock chamber indicated the concrete to be in sound condition except for surface deterioration. Total resurfacing of the lock chamber would be for cosmetic reasons only and was not considered necessary. In order to maintain the function of the lock, concrete resurfacing was required at the ladder and floating mooring bitt recesses. This repair included conventional concrete removal and replacement with the addition of armor to protect the ladders and floating mooring bitts.

321. New exit ladders with protective armor were installed in the lock chamber as shown in Figures 148 through 150. This installation required removal of approximately 150 cu yd of concrete. In most cases, a nominal 14 in. of concrete was removed, except in the immediate vicinity of the ladders and the top corner of the lock wall, where 21 in. of concrete was removed. Concrete removal varied from approximately 50 ft to full face in the lock chamber (61 ft). Concrete removal lines were sawcut to a minimum depth of 3 in. prior to concrete removal. In spite of the sawcut removal line, there was significant overbreak with the explosive blasting used to remove the concrete (Figure 151). The boundaries of these overbreak areas were sawcut to a minimum depth of 3 in., and concrete within these areas was removed by chipping with handheld breakers to a minimum depth of 3 in. (Figure 152). The contractor proposed, and the Corps approved, the use of Weld-Crete as a concrete bonding agent in those overbreak areas in which the replacement concrete was not anchored to the existing wall. The primary factor in selection of this bonding agent was that according to the manufacturer, Larsen Products Corporation, the replacement concrete could be placed up to 10 days after application of the bonding agent with no effect on bond. Thus, the contractor had more than enough time to complete forming of a given area after the bonding agent was applied to the existing concrete.

322. The bonding capacity of Weld-Crete was evaluated at WES as part of the second phase of the dowel-spacing study. When tested according to the Arizona slant shear bond method, the strength of specimens bonded with Weld-Crete and stored under moist conditions was less than 15 percent of that of concrete-to-concrete bond without any bonding agent. This result was

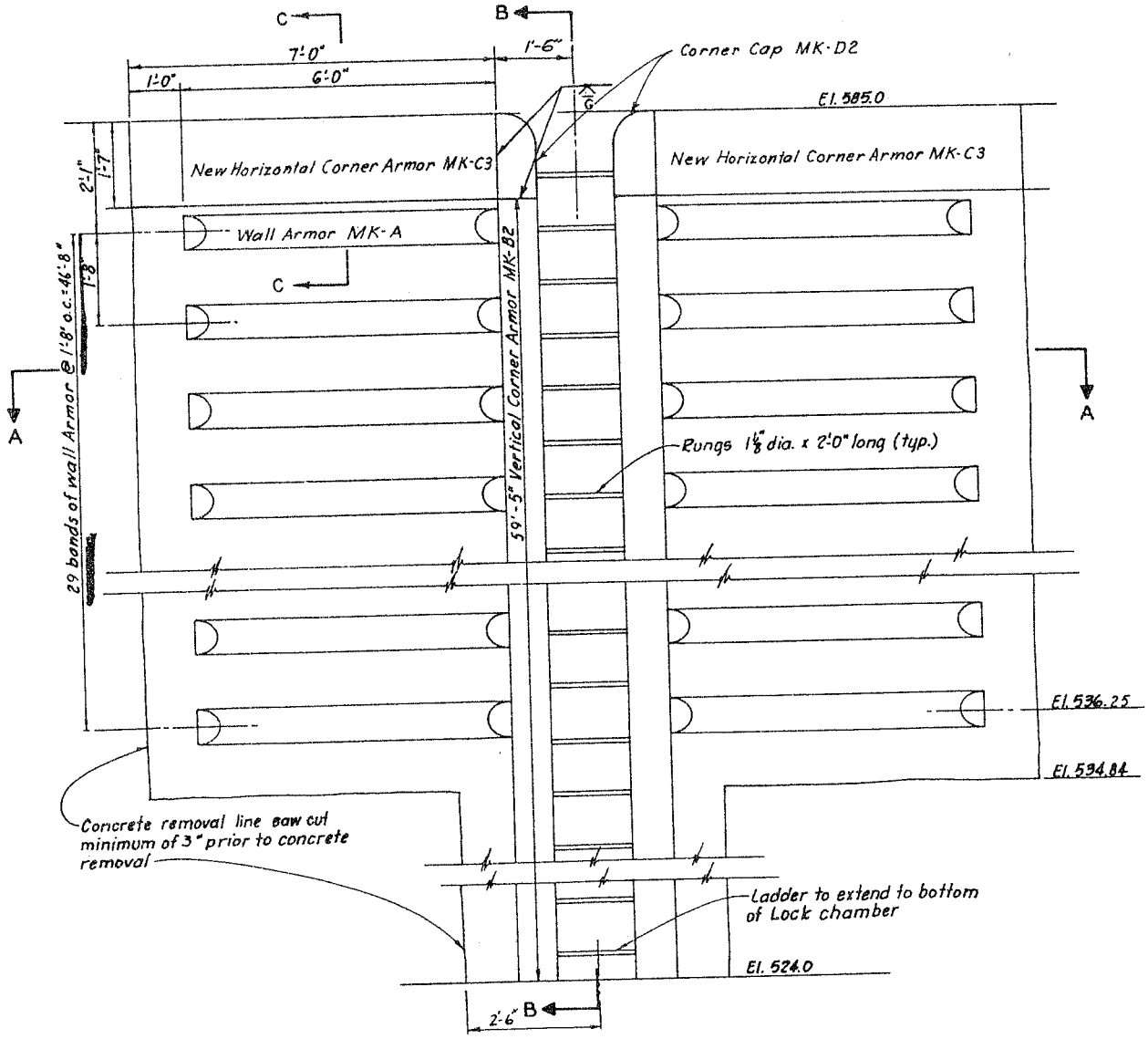


Figure 148. Ladder details, Lockport Lock

attributed to degradation of the polyvinyl acetate bonding agent while the concrete test specimens were stored in a 100 percent relative humidity fog room. Although the manufacturer's literature does not include any limitation on use under moist conditions, it does caution against using the material where hydrostatic pressure is present in the substrate, and it also states that a "wet" type saw should not be used to cut isolation joints. In addition, the literature emphasizes that all joints must be sealed against water penetration. When this information was brought to the attention of Corps project personnel, the use of a bonding agent was discontinued.

323. The contractor elected to use hooked bars, No. 6 on 2-ft centers

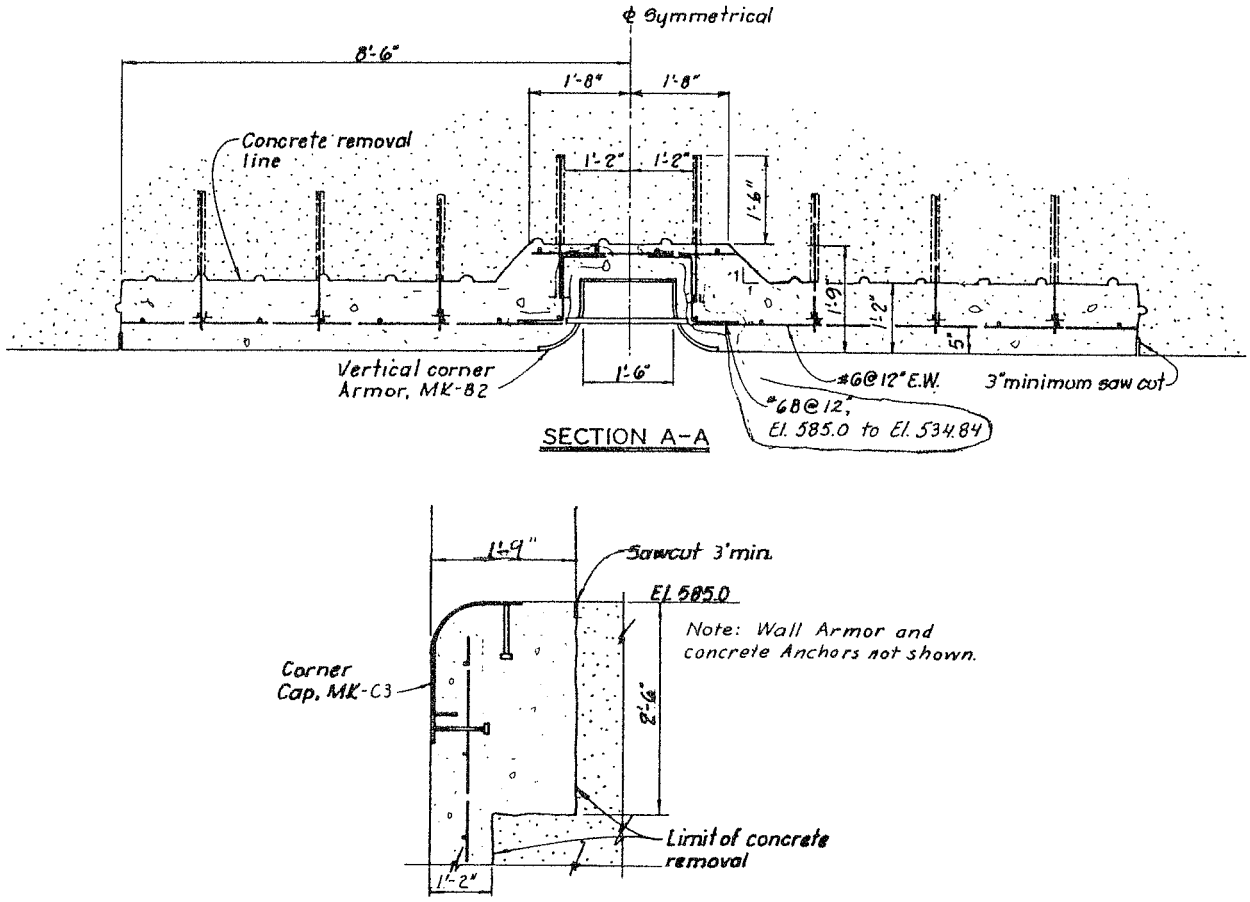
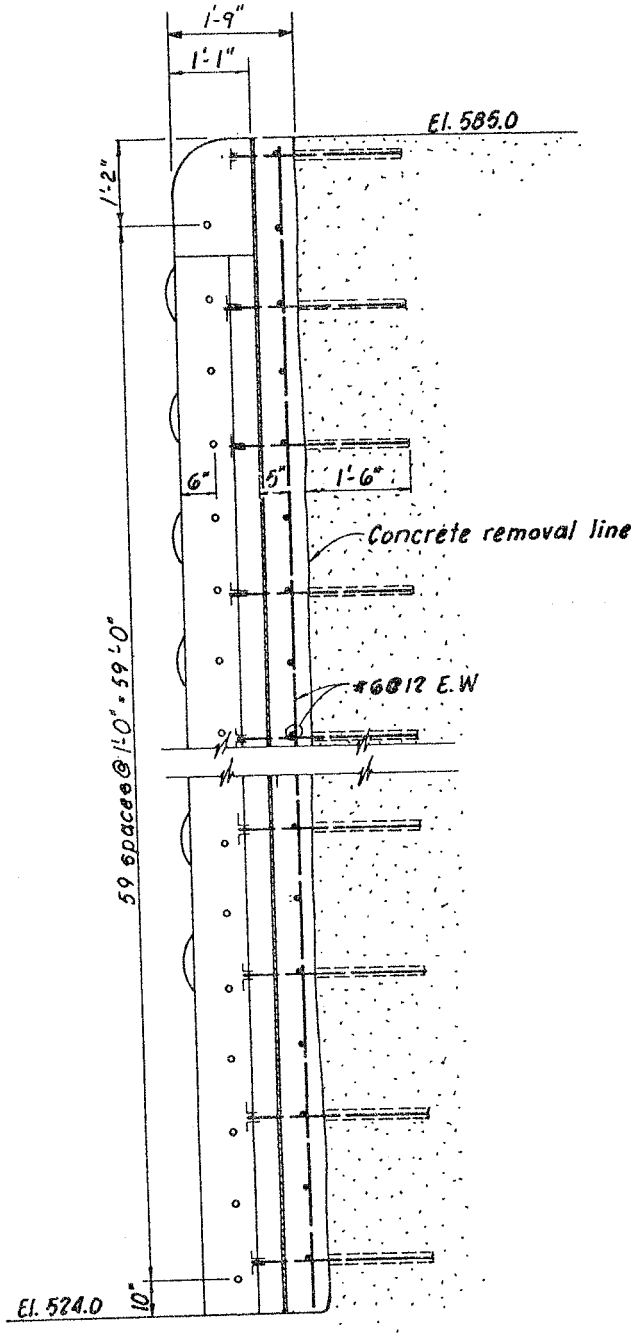


Figure 149. Ladder details of Sections A-A and C-C, Lockport Lock

each way, to anchor the replacement concrete instead of the Richmond anchors shown in the plans (Figure 149). These bars were grouted into 1-in.-diam holes drilled 18 in. into the existing concrete using polyester resin grout. Following installation of the ladders, conventional reinforcing, and armor, the ladder areas were formed in 5-ft lifts (Figure 152). Concrete placement on the landside wall was accomplished by discharging the concrete directly from transit mixers into hoppers filled with flexible hose commonly known as elephant trunks (Figure 153). Concrete on the riverside wall was placed in similar manner except the concrete was transported from the landside wall by concrete bucket and crane. Internal vibrators were used to consolidate the fresh concrete. Forms were usually stripped one day after concrete placement, and a membrane curing compound was applied to formed surfaces. A partially completed ladder section is shown in Figure 154. Some cracking was observed



SECTION B-B

Figure 150. Ladder details of Section B-B, Lockport Lock

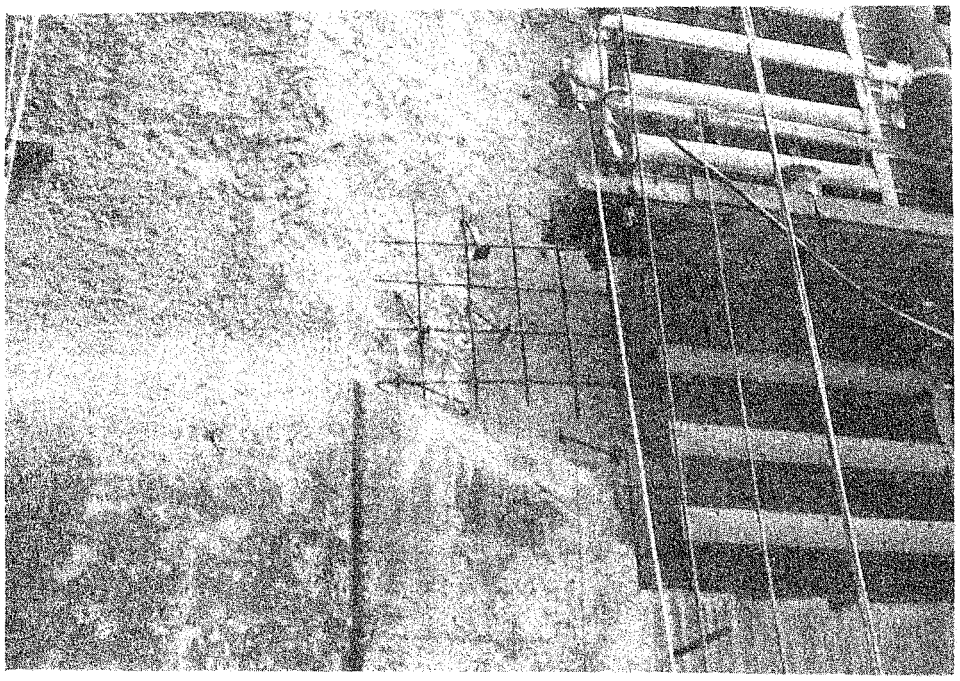


Figure 151. Concrete overbreak during removal, Lockport Lock

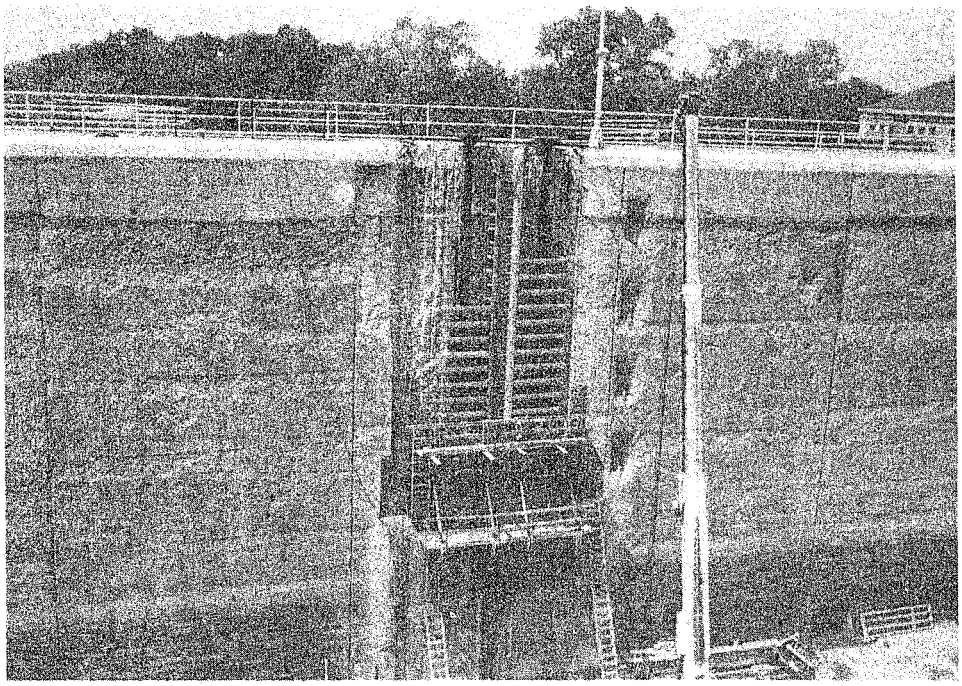


Figure 152. Overbreak area prepared for repair, Lockport Lock

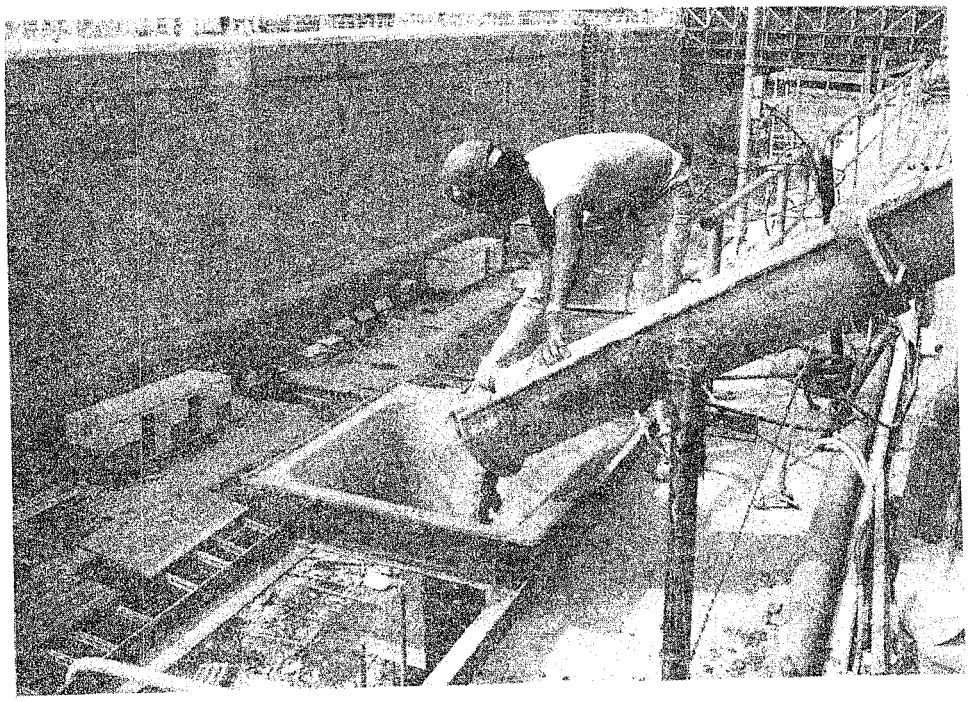


Figure 153. Placing concrete on the landside wall, Lockport Lock

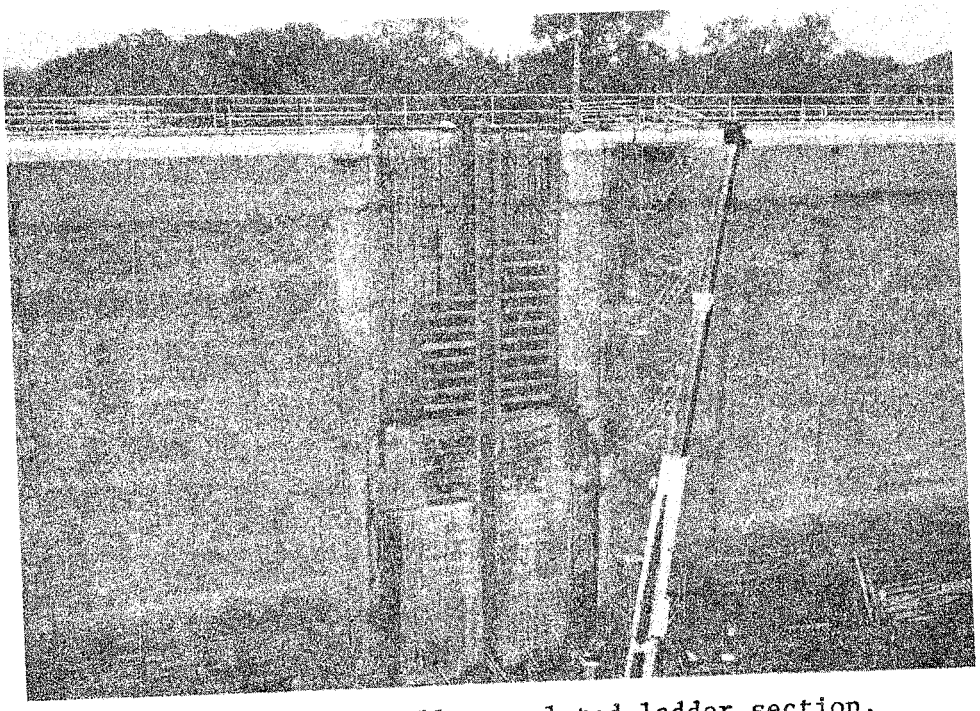


Figure 154. Partially completed ladder section, Lockport Lock

in the replacement concrete, particularly in the lower lifts where armor was not present (Figure 155). The contractor's bid price for ladder installation was approximately $350,000.

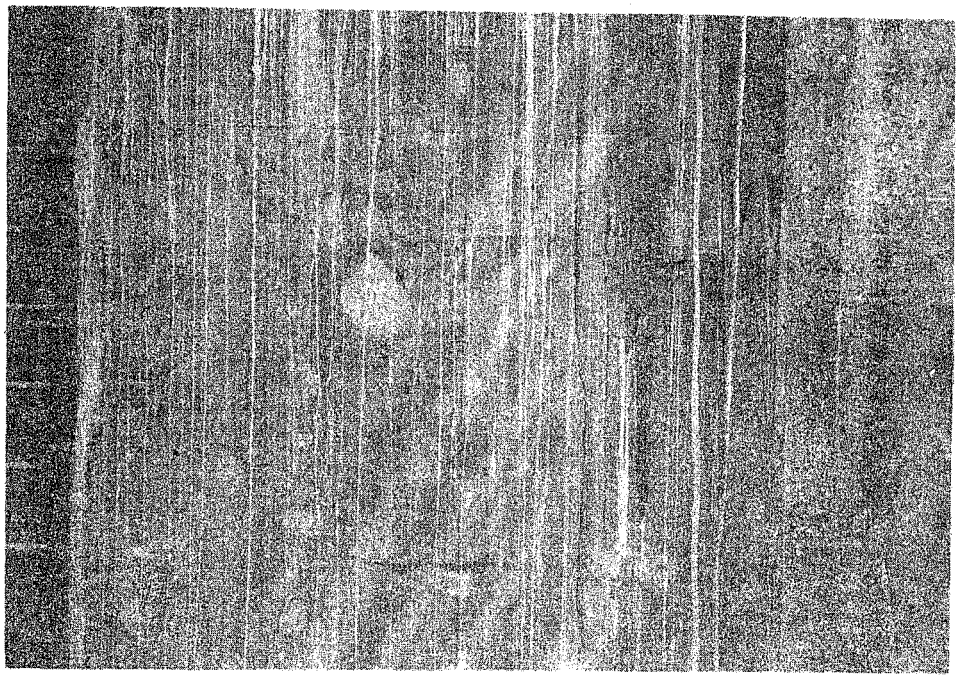
324. Deteriorated concrete around the floating mooring bitts was removed and replaced with armored concrete as shown in Figures 156 through 160. The replacement required removal of approximately 110 cu yd of concrete. Approximately 470 dowels were used to anchor the replacement concrete. Concrete removal, installation of dowels, forming, and concrete placing were accomplished using techniques similar to that previously described for the exit ladders. The contractor's bid price for this work was approximately $240,000.

325. Since the condition survey showed that the maximum depth of concrete deterioration occurred in the lower gate bay, it was completely refaced during the Stage I rehabilitation. The refacing required removal of approximately 335 cu yd of concrete and replacement with new armored concrete. Approximately 960 dowels were required to anchor the replacement concrete (Figure 161). Bid prices for these dowels ranged from $33 to $74 each compared to the Government estimate of $34. A roughly vertical crack which extended from the top of the emptying culvert for a distance of approximately 18 ft was pressure-grouted prior to placement of the new concrete.

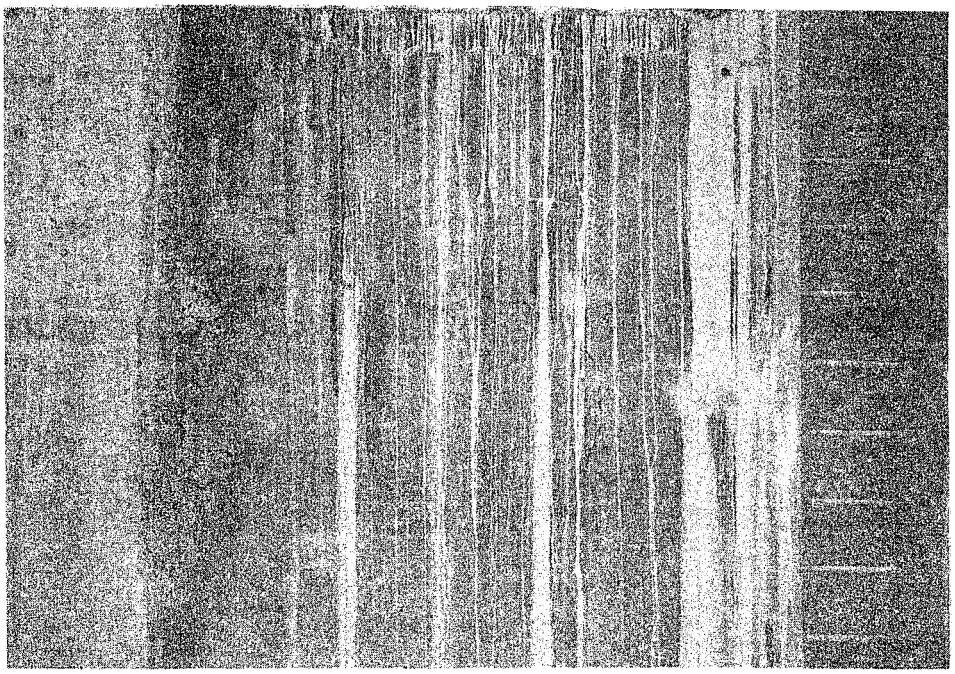
326. The Lockport Lock was reopened to navigation on 29 September 1984. The major work requiring dewatering of the lock was accomplished in 86 days. Current rehabilitation at Lockport Lock includes replacement of the upper lift gates and machinery, work on the culvert valves, river wall resurfacing and stabilization, and lower guide wall and forebay resurfacing.

Brandon Road Lock

327. The Brandon Road Lock and Dam is located at mile 286 of the Illinois Waterway on the Des Plaines River in the city of Joliet, Illinois. The lock is 110 ft wide by 600 ft long with a normal lift of 34 ft (Figure 162). Two sets of miter gates are located at the upstream end, and one set of miter gates is at the downstream end. Brandon Road Lock and Dam was designed by the State of Illinois and constructed by the State and Federal Government. The project was completed in 1933 at a total cost of $4,500,000. Since the



a. Right side of ladder



b. Left side of ladder

Figure 155. Cracking in the replacement concrete, Lockport Lock

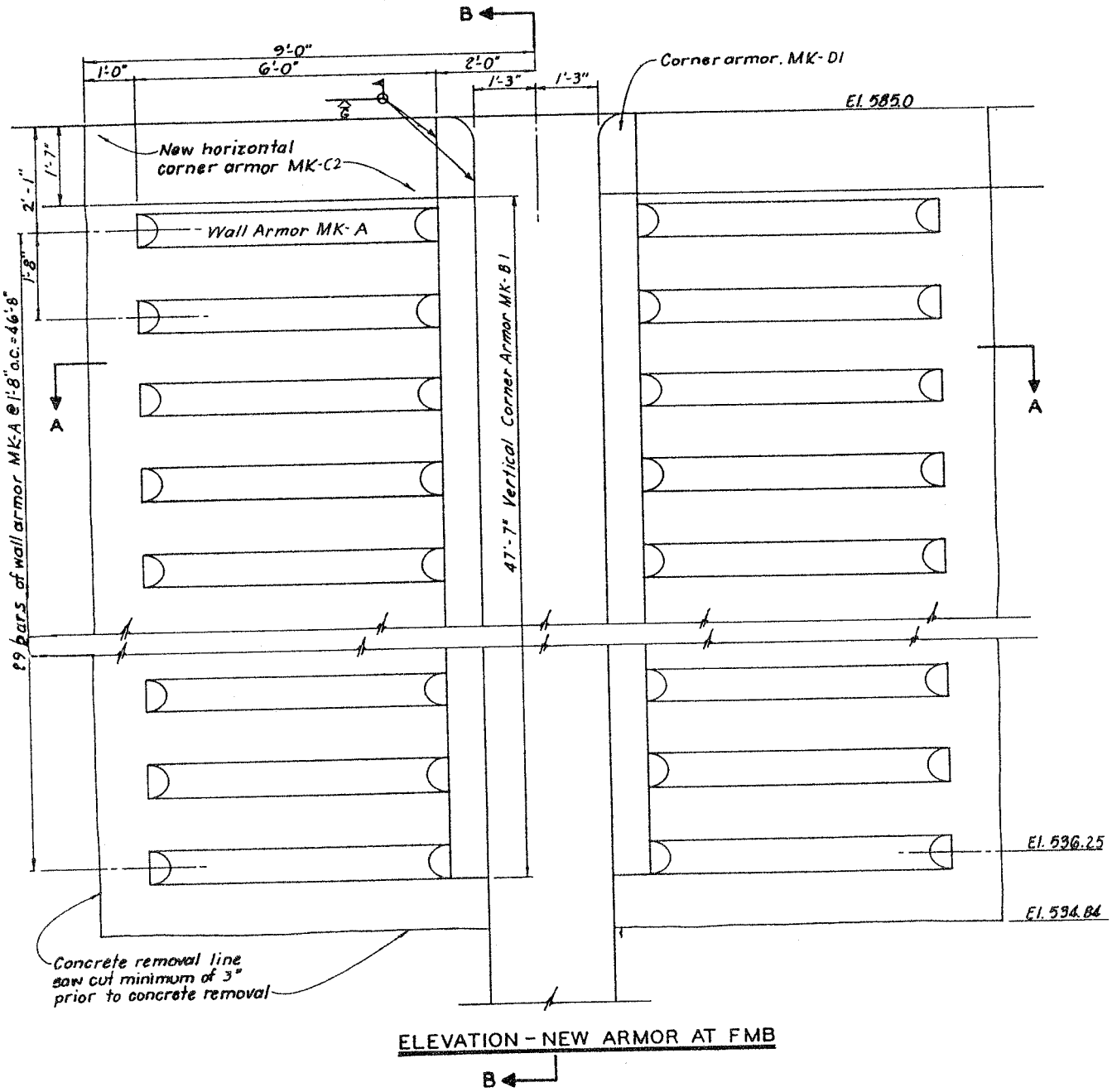


Figure 156. Floating mooring bitt details, Lockport Lock

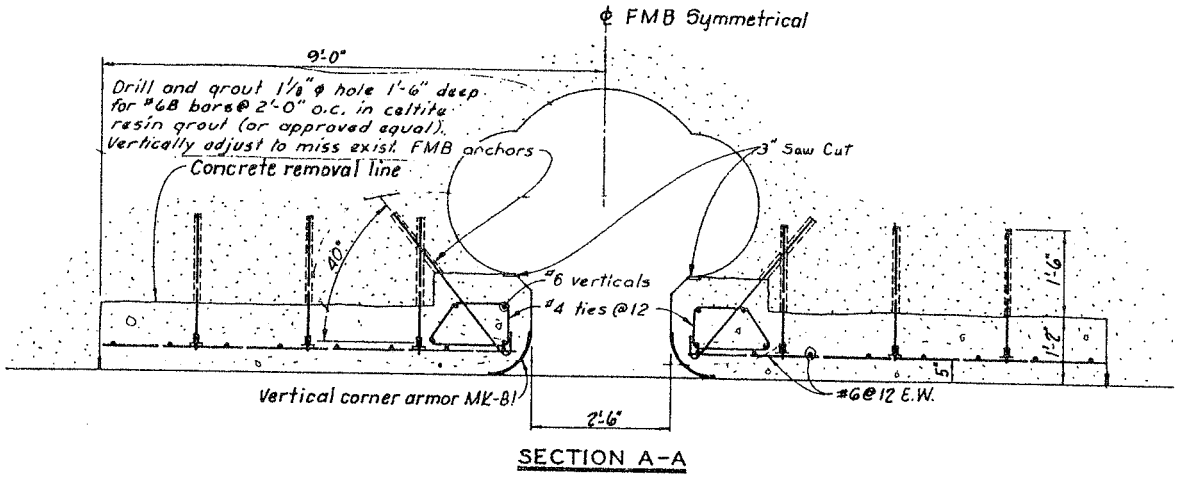


Figure 157. Floating mooring bitt details of Section A-A, Lockport Lock

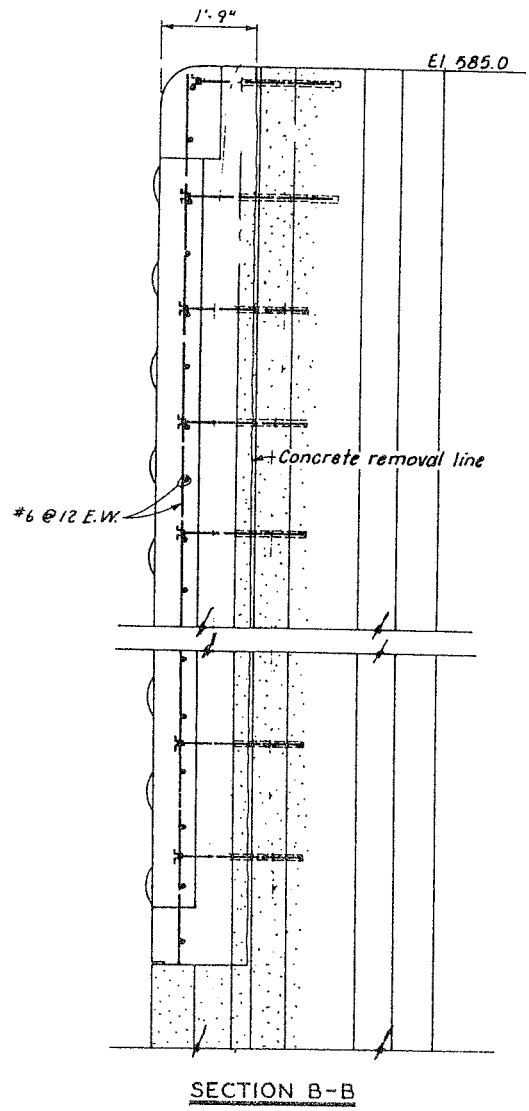


Figure 158. Floating mooring bitt details of Section B-B, Lockport Lock

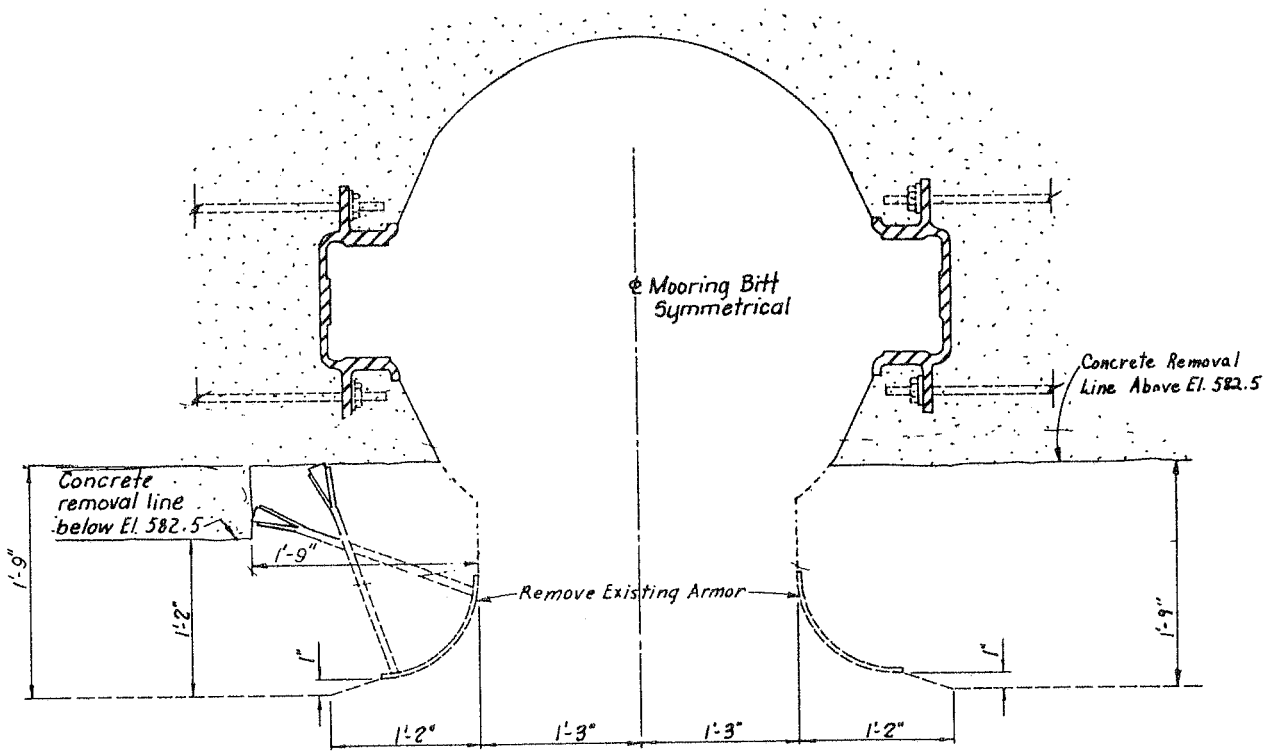
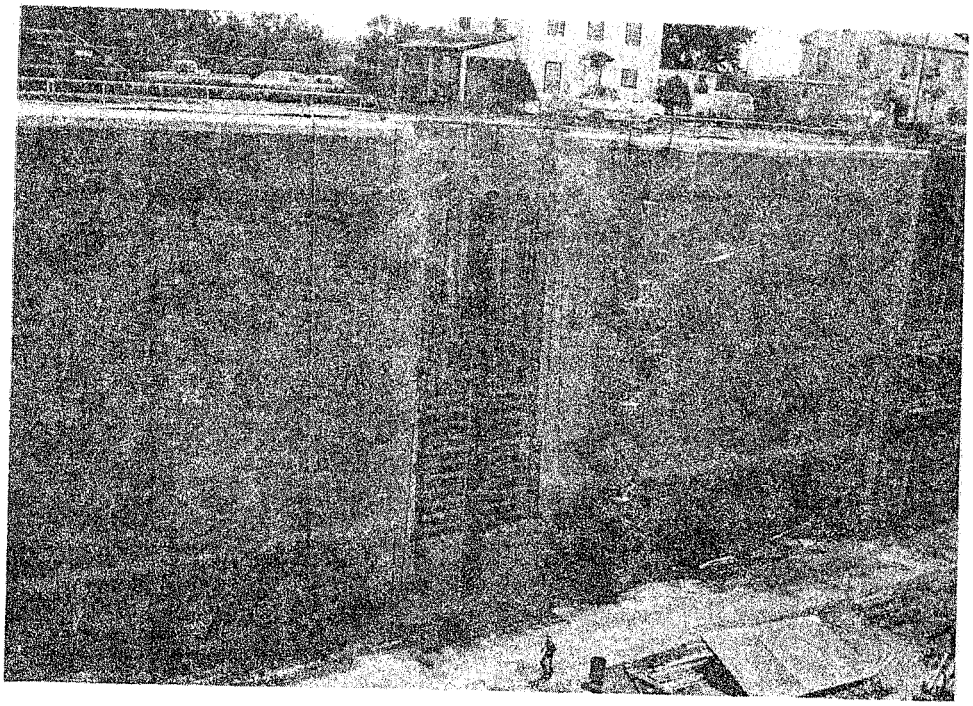
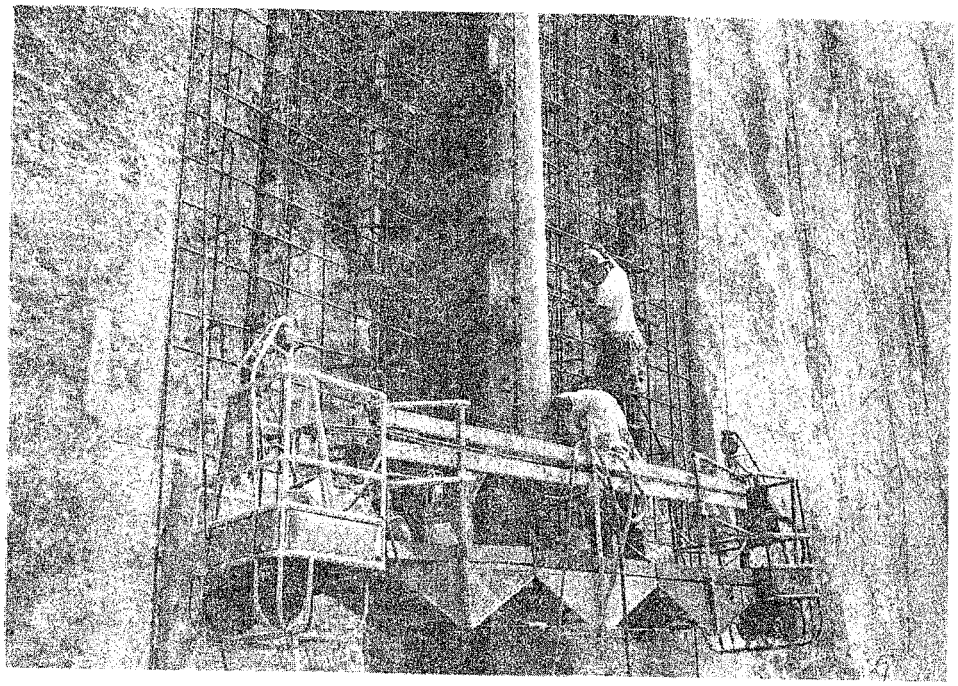


Figure 159. Details of concrete removal floating mooring bitts, Lockport Lock



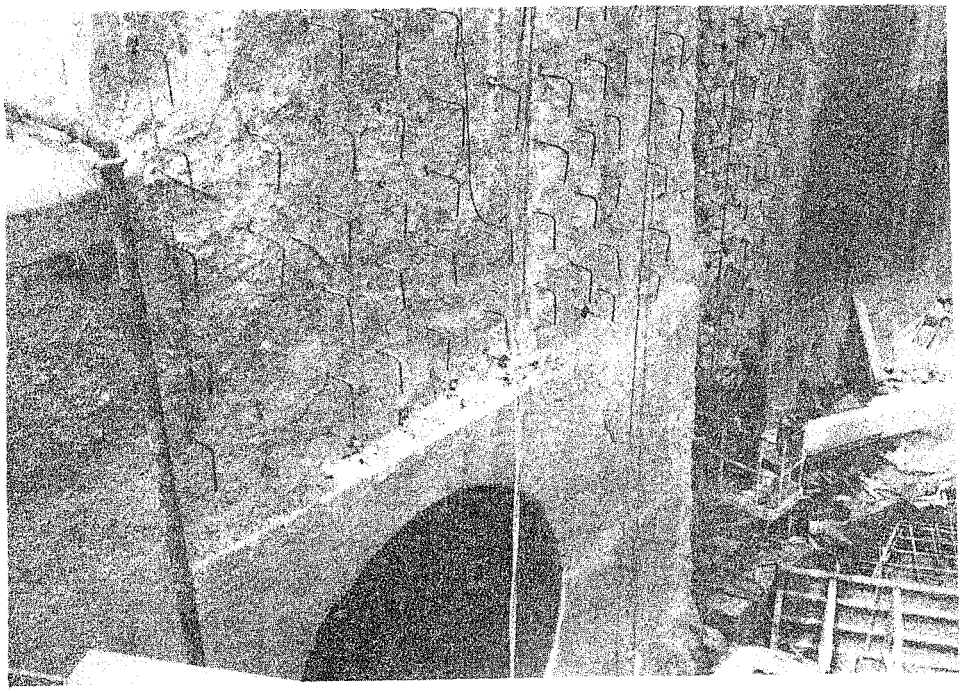
a. Overall view



b. Installing vertical corner armor

Figure 160. Replacing concrete around a floating mooring bitt, Lockport Lock

a. Installation of dowels



b. Crack in top of culvert to be grouted

Figure 161. Rehabilitation of lower gate bay, Lockport Lock

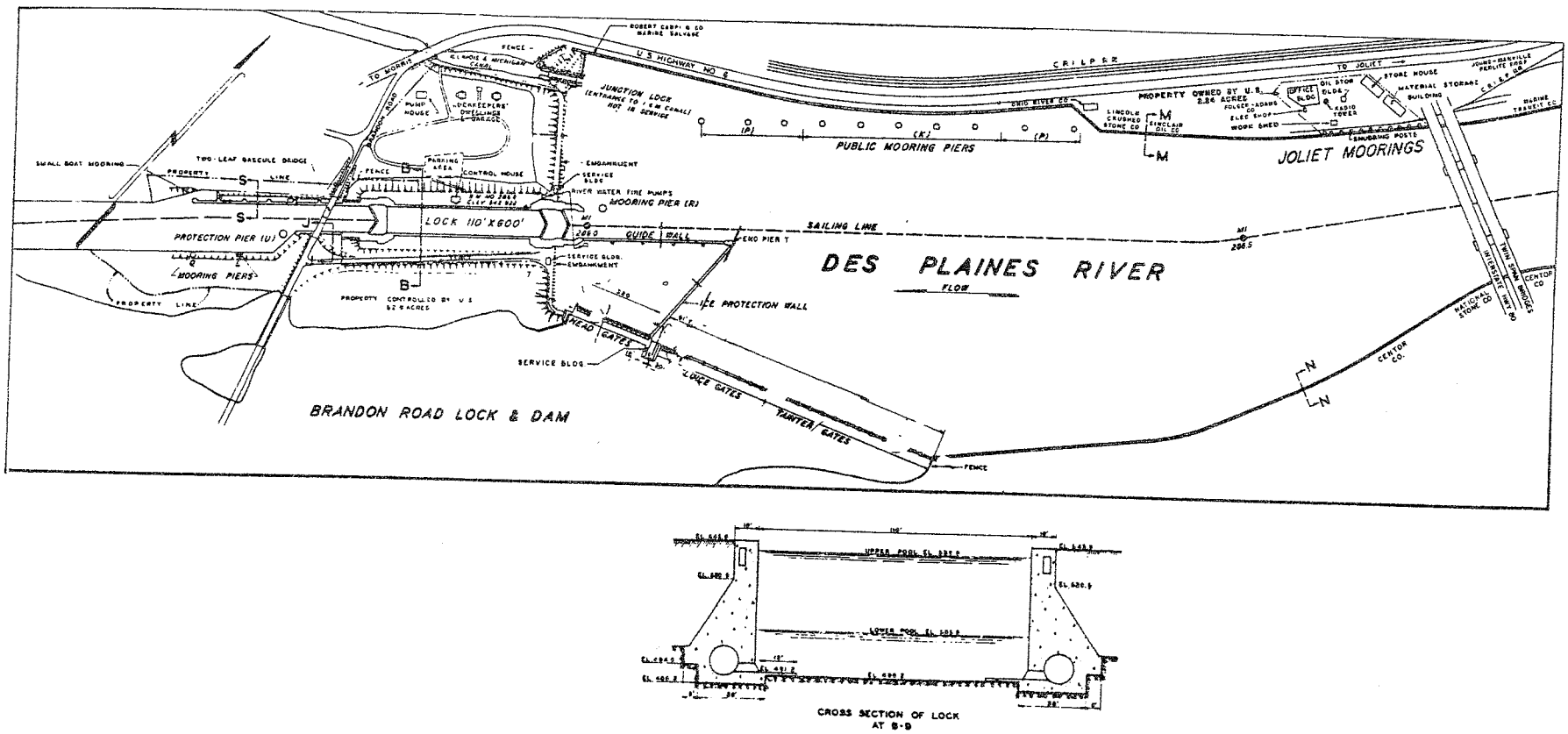


Figure 162. Plan and typical section, Brandon Road Lock

original construction, the following major rehabilitation has been performed:

a. The upstream faces of the tainter gate piers were repaired in 1958.

b. Floating mooring bitts were installed in 1965.

c. The downstream protection pier was repaired in 1966 and 1967.

d. The top of lock walls were resurfaced with 4 in. of new concrete in 1967.

e. New hydraulic culvert valve gate machinery was installed in 1969.

f. Trash racks and trash rakes were installed in the lock-filling conduits in 1972.

g. Repairs were made to the boiler house and ice chute pier in 1975.

h. Dam stabilization with prestressed anchors, ice chute conversion to an overflow section, scour protection downstream of the tainter and sluice gates, and headgate repairs were completed in 1980.

328. The basis for design of the Stage I Brandon Road Lock Rehabilitation is contained in Design Memorandum No. 1 (Rock Island District 1983). The design aspects followed closely the design of similar features at Lockport. However, the requirements were considerably different. Stability was only a concern at the relatively thin lower guide wall. The miter gates were also considered repairable, and replacement was not necessary. The inside face of the lock walls was more deeply deteriorated than at Lockport, so refacing and armoring of the lock walls was decided upon rather than selective placement as at Lockport.

329. James McHugh Construction Company, Chicago, Illinois, was low bidder for the Stage I rehabilitation of Brandon Road Lock. An abstract of the bids submitted is shown in Appendix A. Bids ranged from $8.0 to $12.3 million compared to the Government estimate of $14.8 million. The major work items under this contract were concrete resurfacing of the entire lock chamber, rehabilitation of the upper and lower miter gates, resurfacing of the upper guide wall, stabilization and rebuilding of the lower guide wall, and complete replacement of the lock electrical distribution system.

330. Resurfacing of the lock chamber monoliths involved removing approximately 2,700 cu yd of concrete and replacing it with new concrete (Figures 163 and 164). Prior to closure of the lock, the contractor started

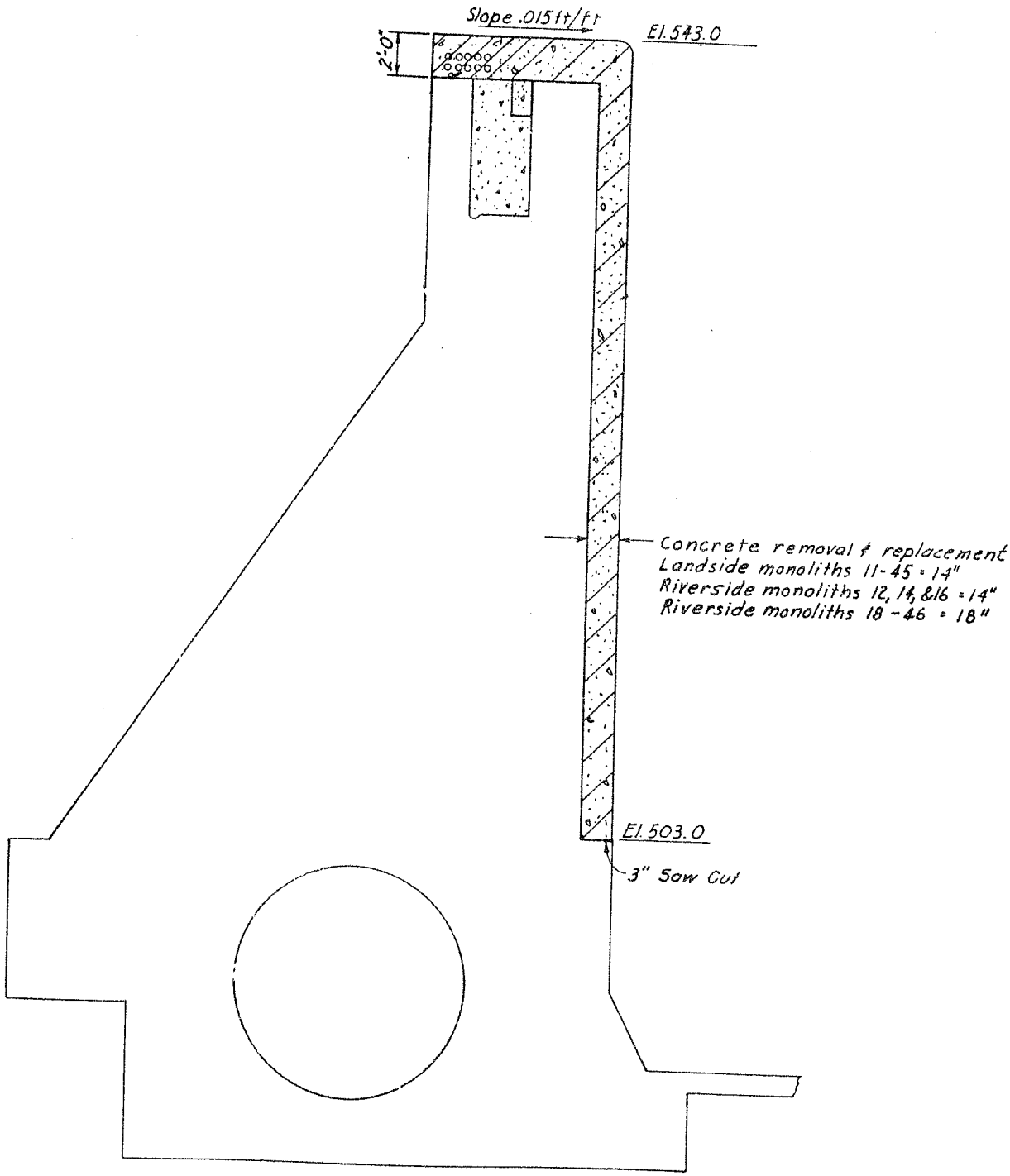


Figure 163. Resurfacing details, chamber monoliths, Brandon Road Lock

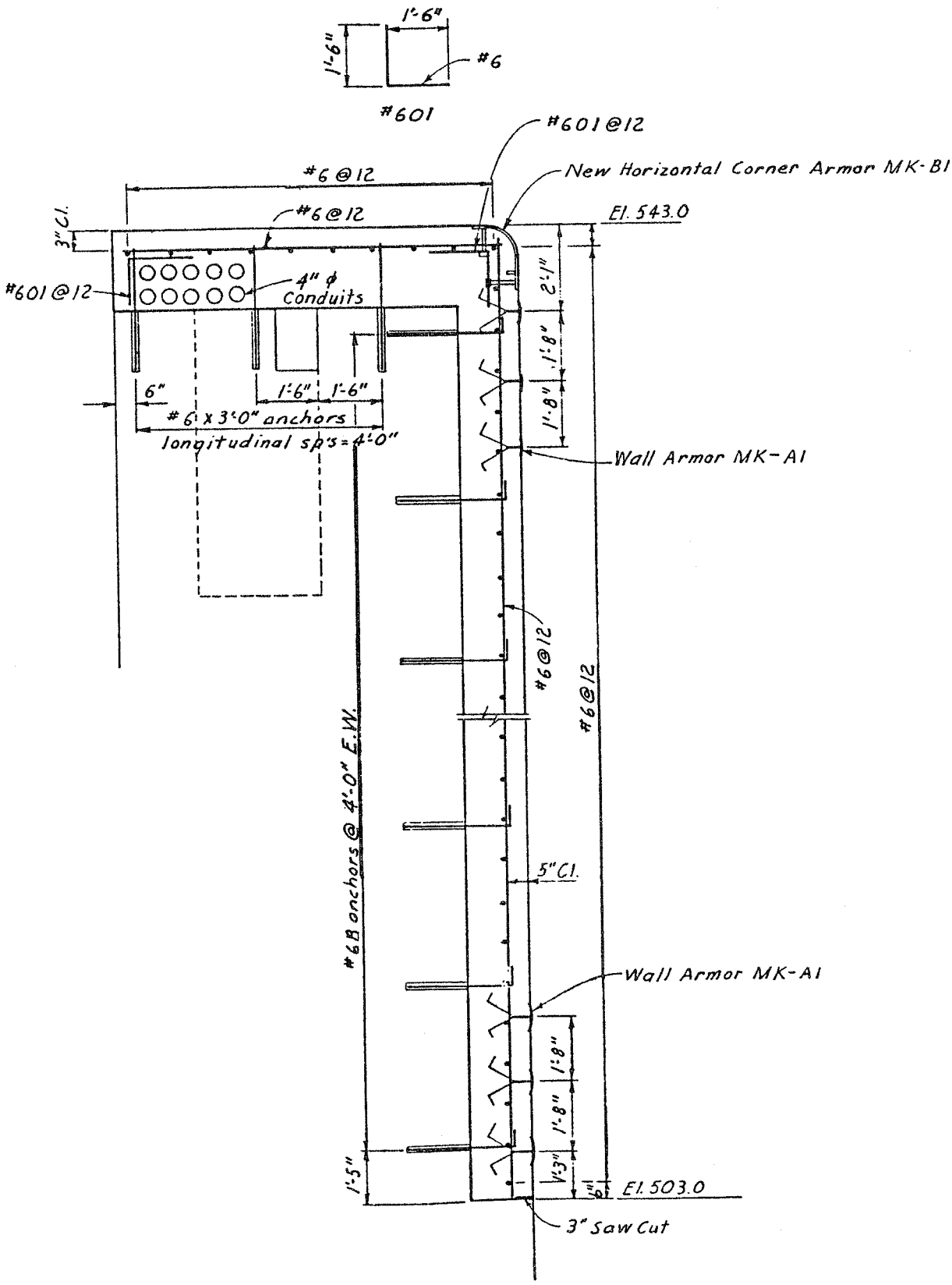
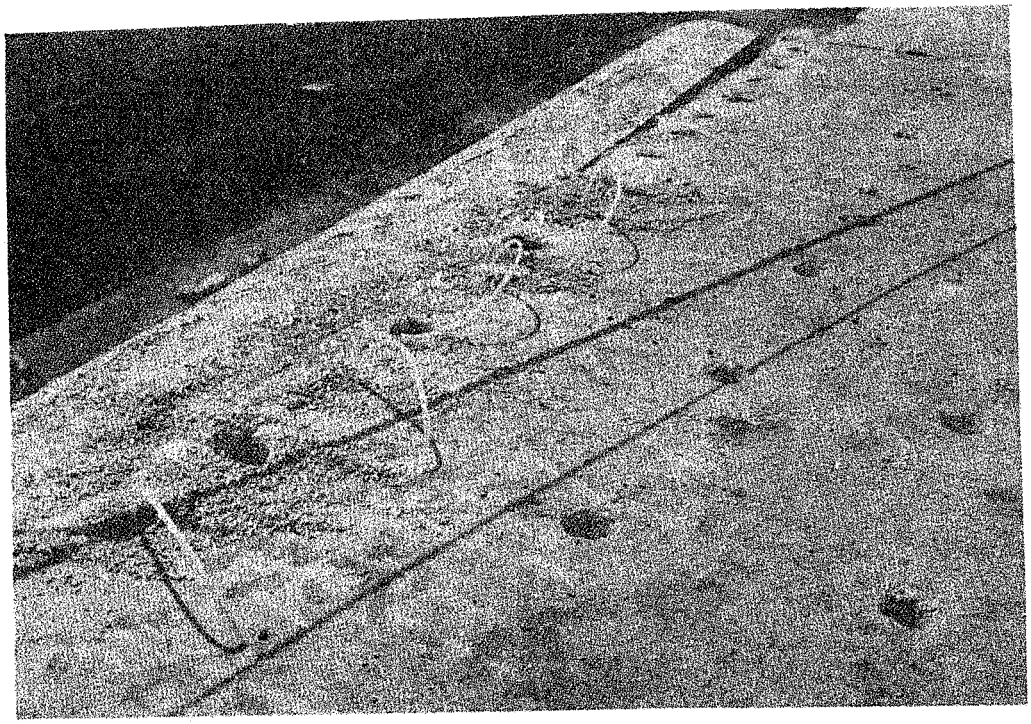


Figure 164. Reinforcing details, chamber monolith resurfacing, Brandon Road Lock

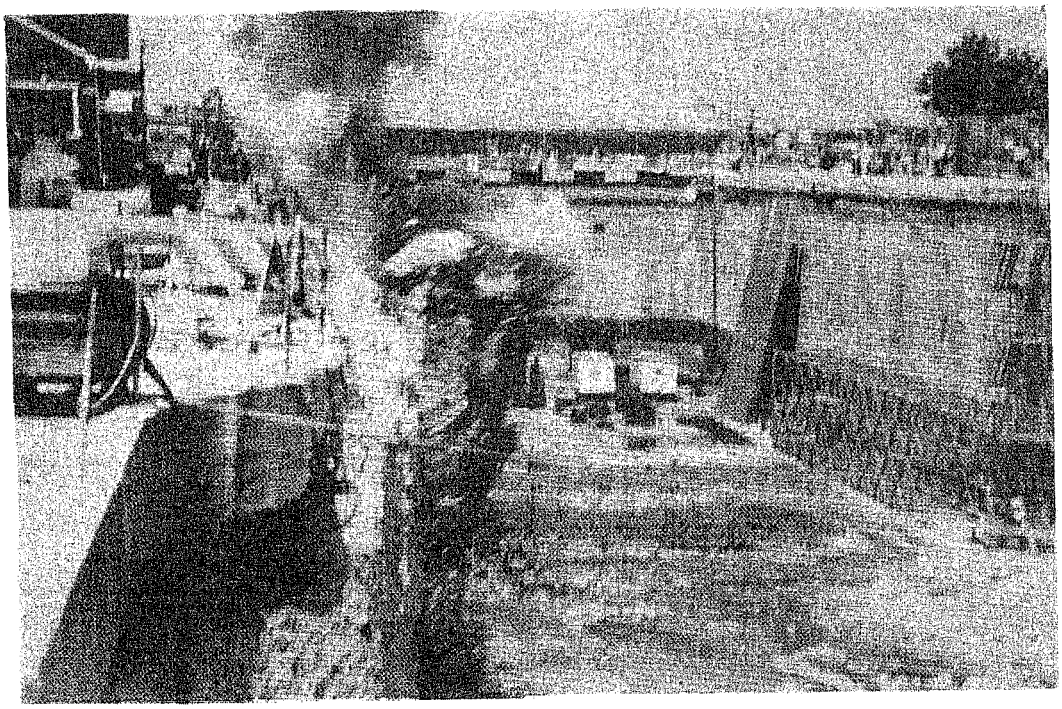
preparations for concrete removal by line drilling a series of 2-1/2-in.-diam holes on 9-in. centers along the top of the lock wall. These vertical holes were located between 15 and 24 in. back from the chamber face and were drilled to a depth of 40 ft. Brandon Road Lock was closed to navigation on 5 July 1984, coinciding with the closure at Lockport Lock. Immediately after the lock was dewatered, a horizontal saw cut 7 in. deep was made along the lower work line, approximately 2 ft below low-water pool, to control overbreak during concrete removal. Also, the bottom 2 ft of the holes was stemmed with sand to prevent overbreak. The contractor then inserted 100-grain/ft detonating cord with blasting caps into the holes, plugged the top of the holes to prevent loss of energy, and detonated the explosives with an electrical charge (Figure 165). Blasting was restricted to one 30-ft monolith at a time.

331. Following blasting, the walls were scaled to remove loose concrete still clinging to the wall surfaces (Figure 166). This scaling, normally done by manual processes such as labor crews working from scissor lifts or scaffolding with handheld hammers and chipping tools, is a very costly and time-consuming operation. The Cutter Boom, a modified piece of mining equipment having a rotary head cutter with a series of carbide-tipped teeth that grind away the concrete (Figures 167 and 168), was used to scale the walls at Brandon Road Lock. The efficiency of this equipment probably contributed to the fact that the contractor's bid price for removal of concrete in the lock chamber was $162 per cu yd as compared to the Government estimate of $771 per cu yd.

332. The Cutter Boom used by the contractor at Brandon Road was provided by Excavation and Tunneling Equipment Corporation (ETE), State College, Pennsylvania. ETE provides Cutter Booms from 55 to 215 hp that can be custom fitted to most common backhoe excavators. The 165-hp version used at Brandon Road was mounted on a Caterpillar Model 235 excavator. The cutter is powered by a 165-hp electric motor which runs on 440 volts AC and draws approximately 200 amps under load. A 165-hp rated transmission reduces the output speed to the cutter head assembly. The 26-in.-diam cutter head, which has 108 cutter bits, rotates at 82 revolutions per minute. The electric motor is water cooled with an open loop cooling system that uses spray nozzles directed toward the cutter head for dust suppression. The main electrical enclosure is mounted on the swing frame, and the operator's control station is mounted in the cab area. A specially designed structural frame for mounting the electric



a. Primacord inserted into predrilled holes



b. Detonation

Figure 165. Concrete removal by blasting, Brandon Road Lock

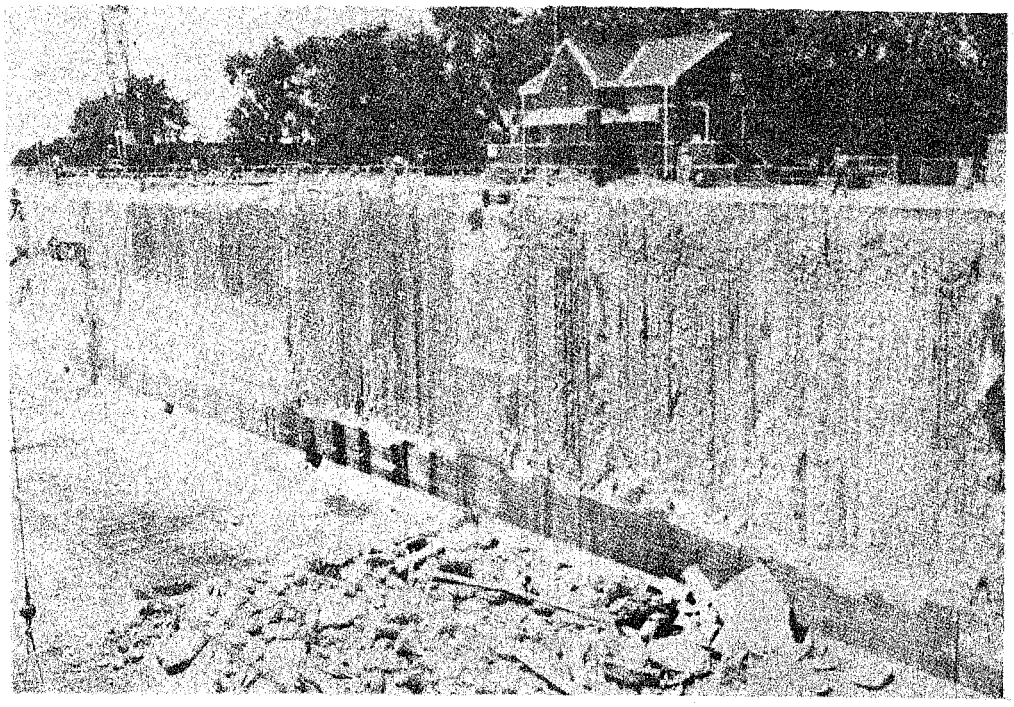
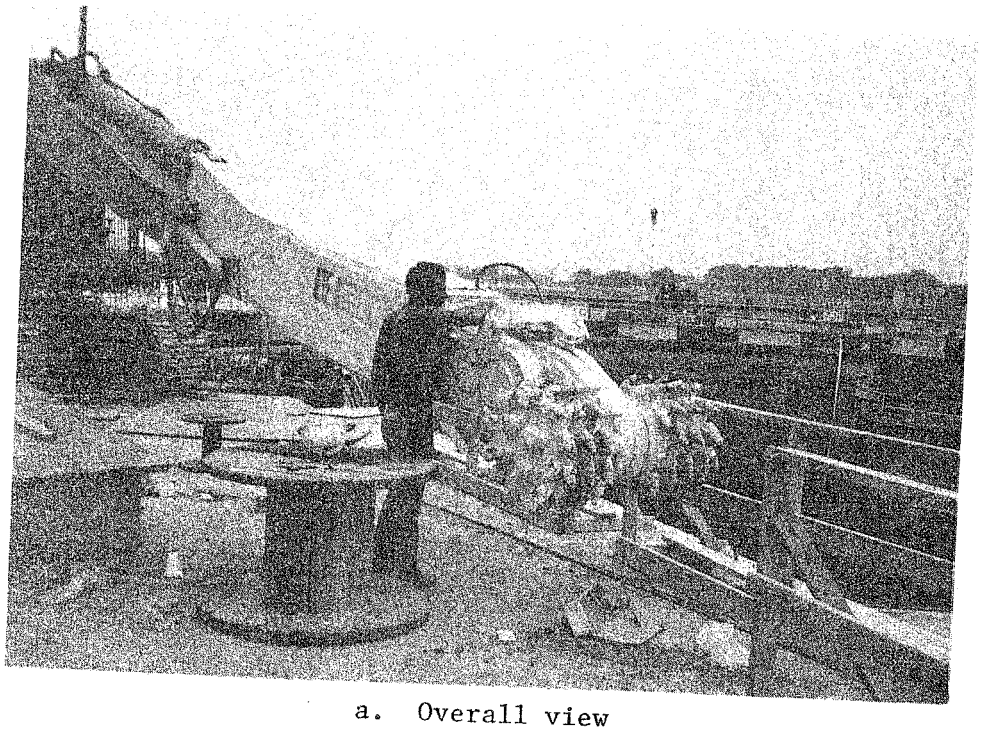
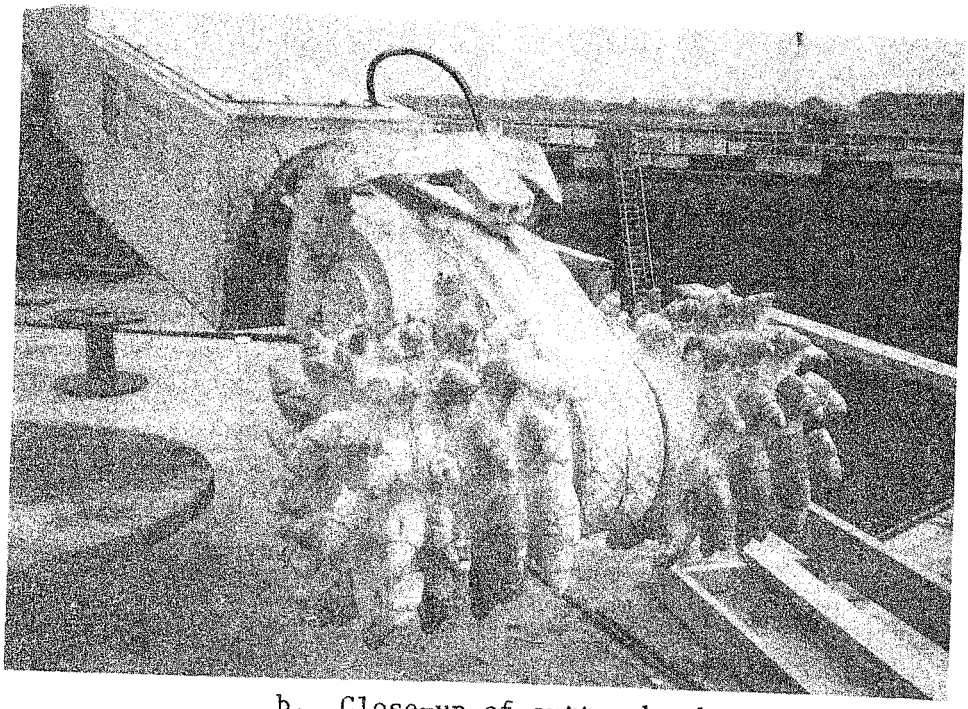


Figure 166. Typical concrete surface conditions following blasting, Brandon Road Lock

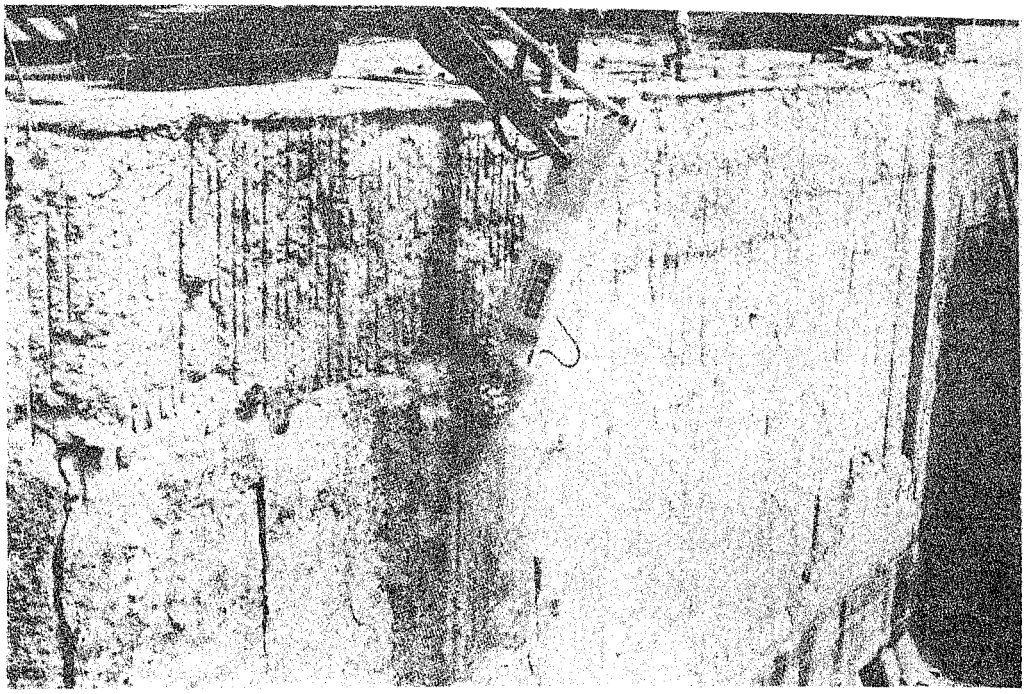


a. Overall view

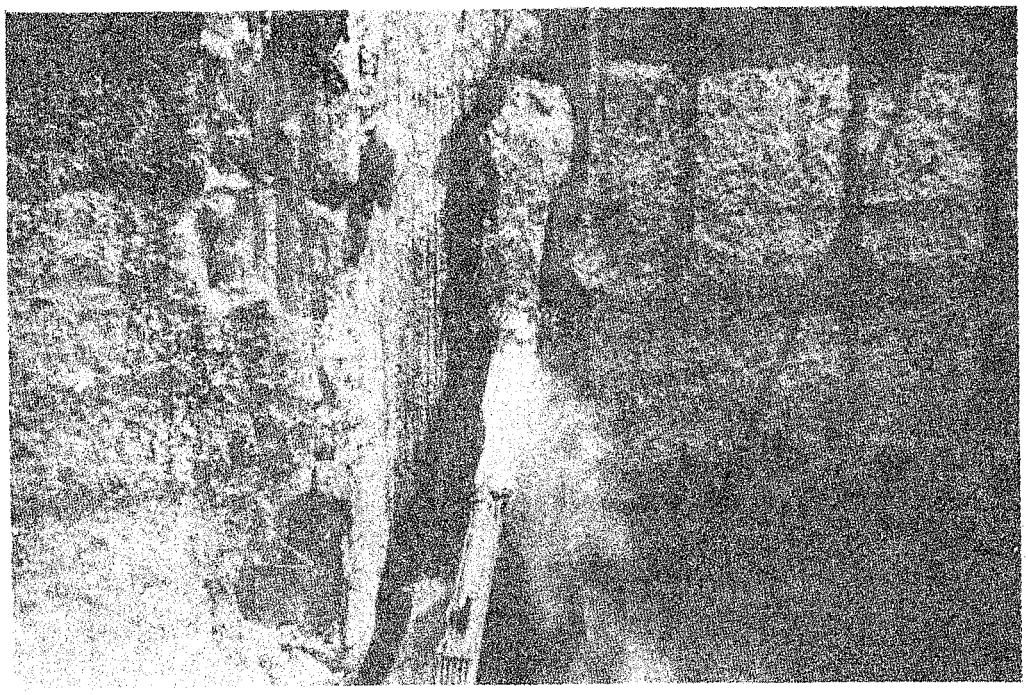


b. Close-up of cutter head

Figure 167. Cutter Boom used in concrete removal, Brandon Road Lock



a. Working from the top of the lock wall



b. Working from the chamber floor to remove concrete in the miter gate recess

Figure 168. Concrete removal using the Cutter Boom, Brandon Road Lock

motor and transmission is fastened to the Model 235 excavator using the existing stick pins. Thus, the cutterhead can reach down a vertical surface approximately 17 ft below the machine platform and nearly 30 ft up a vertical wall.

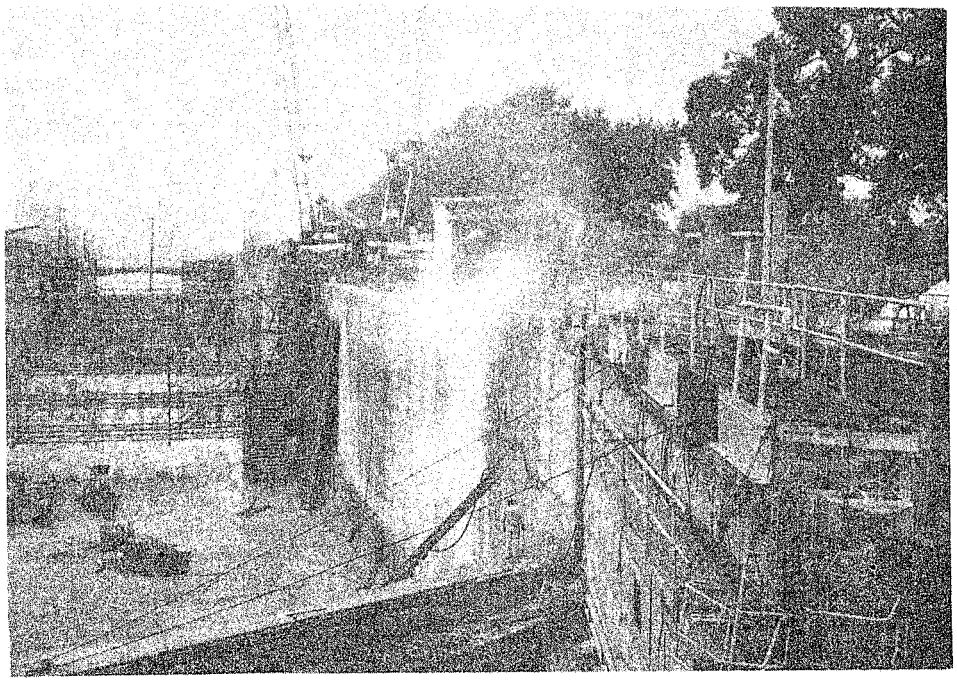
333. In addition to the scaling operation, the Cutter Boom was very effective in grinding out soft pockets or honeycombed areas in the concrete structure as detected by Corps and contractor quality control inspectors sounding the walls. Also, deeper cuts were easily made by the Cutter Boom to accommodate lock chamber appurtenances such as exit ladders and line hook fixtures (Figures 169 and 170) that required deeper embedment in the existing structure than was afforded by the standard removal line.

334. The concrete in the lock face did not include reinforcing steel; however, in some areas, a considerable number of steel form ties protruded from the wall after blasting. The replaceable carbide cutting bits on the cutter head clipped most of these ties off in the normal scaling operation. The newer sidewalk over the top of the lock walls included steel wire mesh. In the initial phases of concrete removal, contractor personnel were placing blasting mats on top of the structure and using small explosive charges to fracture the sidewalk and 2-ft-depth concrete cap that had to be removed. After observing how effective the cutter head was in clipping off steel form ties and grinding up the fractured sidewalk and concrete cap, the contractor experimented with grinding the structure cap and sidewalk without explosive fracturing and found the use of explosives to be unnecessary.

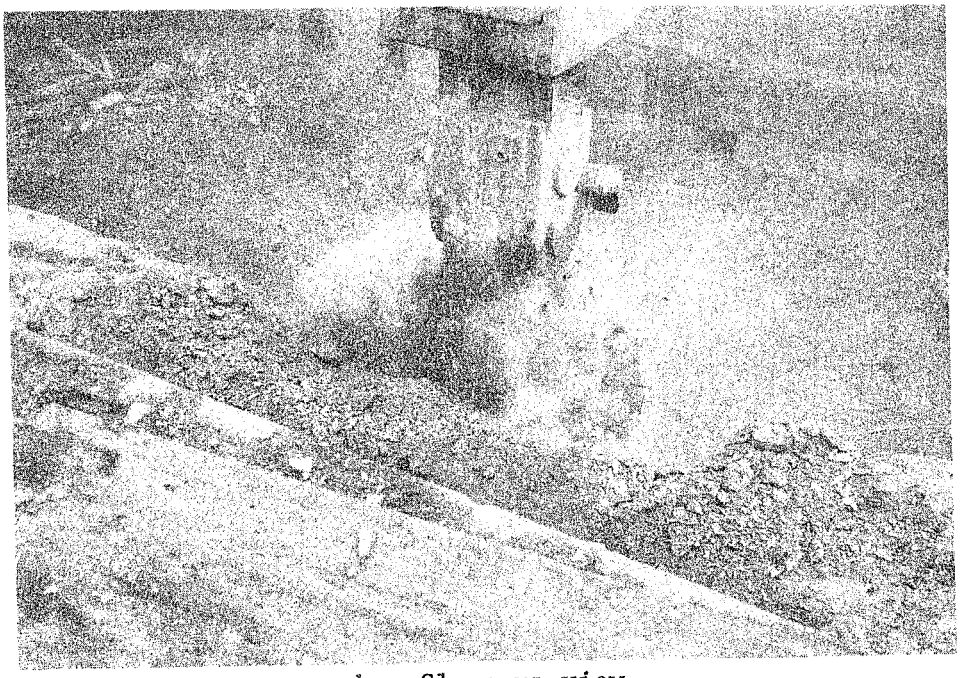
335. While actual cost comparisons of manual removal of concrete versus removal by the Cutter Boom were not available, the Cutter Boom was, according to Corps project personnel, an obvious success in the compressed schedule application at Brandon Road. According to ETE, the performance characteristics of the 165-hp Cutter Boom working in concrete are as follows:

Material	Rates Net/Hour Cutting Time	Bit Cost	Maintenance Costs/Year Single Shift Operation
Concrete 5,000-6,000 psi	10-15 yd^3/hr	50¢/yd^3	$15,000

Other advantages include well-defined limits of concrete removal, relatively small, easily handled concrete debris, and simplicity of operation. According to the Cutter Boom operator, "anyone that can operate a backhoe can operate

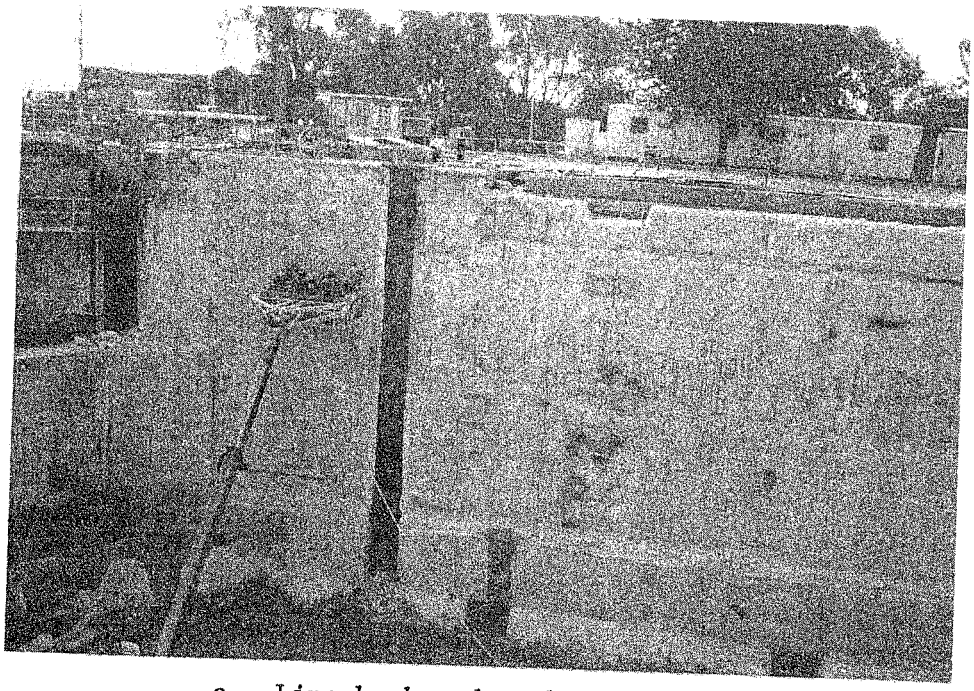


a. Overall view

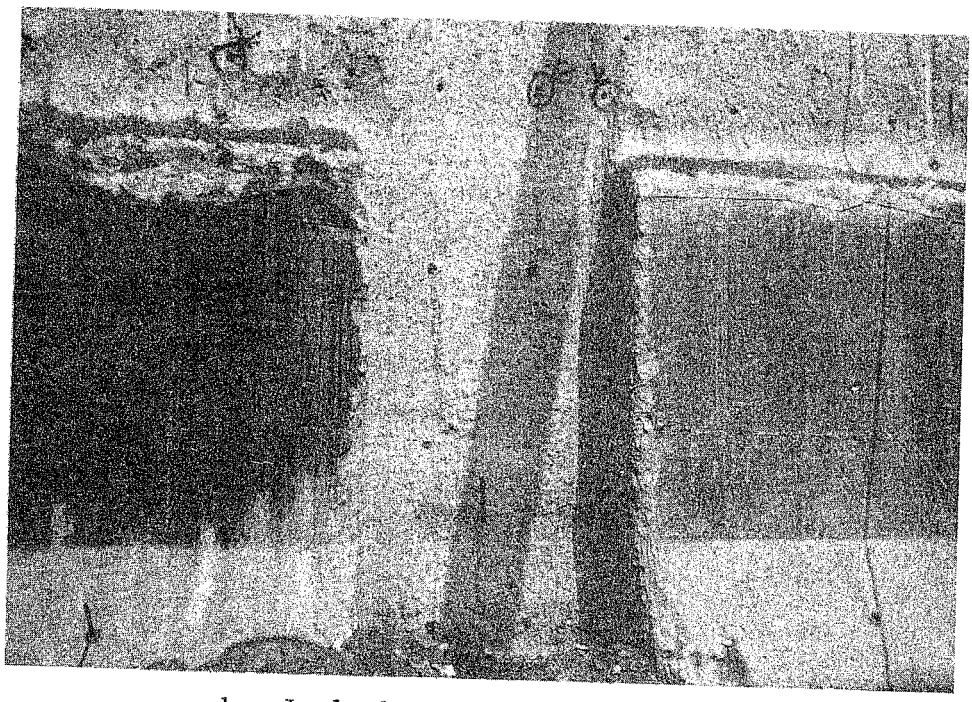


b. Close-up view

Figure 169. Cutting an embedment area for a snub post, Brandon Road Lock



a. Line hook and snub post cuts



b. Lock chamber exit ladder cut

Figure 170. Completed cuts for embedment of line hook, snub post, and exit ladder, Brandon Road Lock

the Cutter Boom." The Cutter Boom proved to be such an efficient machine that on other projects presently being rehabilitated, the concrete is being ground back to the required removal line without any preliminary work such as predrilling, saw cutting, or blasting.

336. Limitations or disadvantages of the Cutter Boom include large electric power demand, limited mobility from dragging a heavy power cable around a congested construction site, and dust. The water spray nozzles did not appear adequate for dust suppression, thus making it "difficult or impossible to work downwind of the Cutter Boom," according to contractor personnel. Also, it is impossible for the operator to see the cutter head when working from the top of the lock wall. So far all Cutter Booms supplied by ETE have been electrically driven, but hydraulically driven cutter heads can be made available. The latter type, according to contractor personnel, would significantly increase the mobility of the system.

337. Once concrete removal was completed, concrete surfaces were cleaned with a high-pressure water blaster (Figure 171). Holes for dowels to anchor the replacement concrete were drilled with a hydraulically powered gang drill as shown in Figure 172. In general, dowels were installed on 2-ft centers around the perimeter of monoliths and 4 ft on centers each way in the

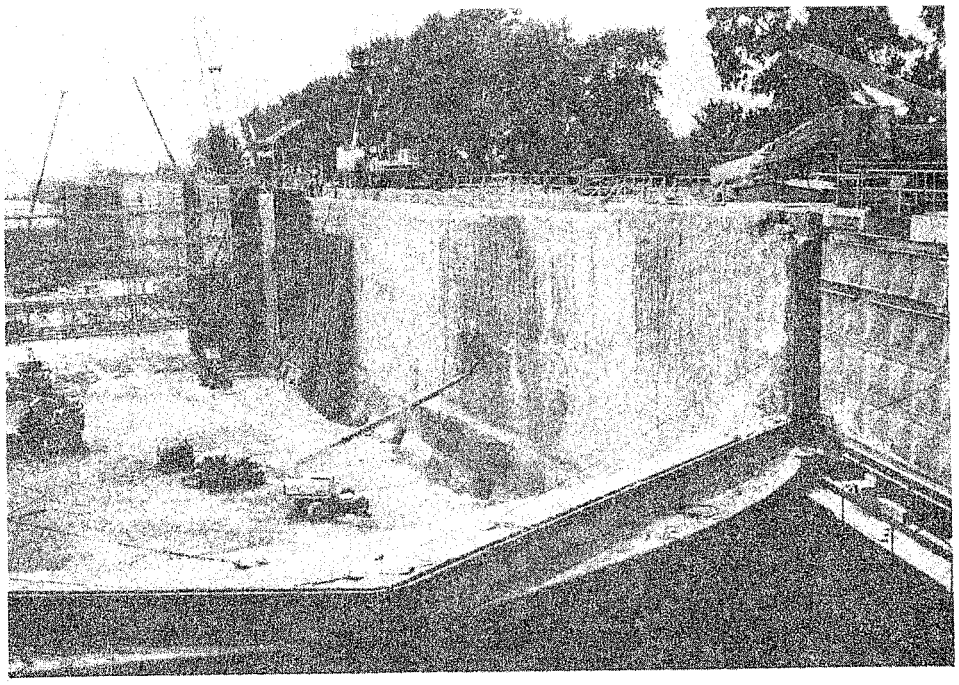


Figure 171. Water blast cleaning of concrete surface, Brandon Road Lock

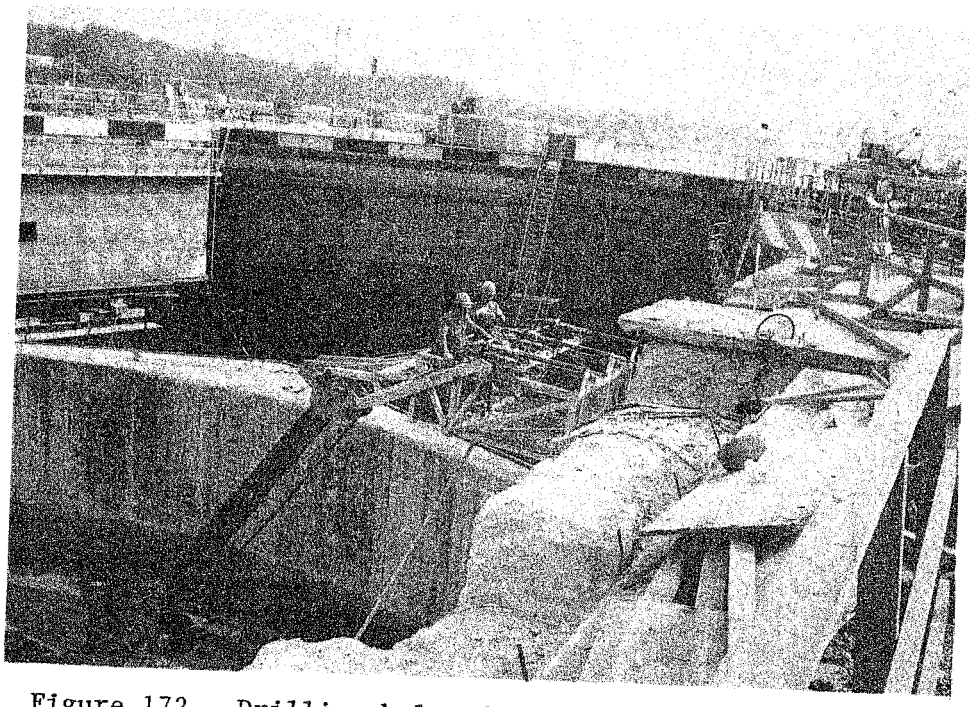


Figure 172. Drilling holes for dowels, Brandon Rock Lock

remainder of the monoliths. Instead of the hooked bars shown in the plans (Figure 164), the contractor elected to use straight bars with a nut welded on the end. Dowels were grouted with FASLOC, a polyester resin grout manufactured by Dupont. Selected anchors, two or three per monolith, were load tested to 20 kips with the use of a hollow core hydraulic ram (Figure 173). Approximately 3,800 dowels were required to anchor the replacement concrete in the lock chamber resurfacing. The contractor's bid price for these concrete anchors was $15 each as compared to the Government estimate of $44 each.

338. Wall armor and concrete reinforcement, No. 6 bars on 12-in. centers each way, were installed on the concrete form prior to placing the form on the lock wall (Figures 174 and 175). A truss system was used to span the lock chamber, thus supporting forms on opposing monoliths of each wall simultaneously (Figure 176). Replacement concrete was placed in a single 40-ft lift for each 30-ft-wide monolith. Approximately 85 and 100 cu yd of Type C concrete was required for landside and riverside monolith placements, respectively. Concrete materials, mixture proportions, and supplier were the same as previously reported for Lockport Lock (paragraph 314). Concrete for resurfacing the vertical walls was discharged into hoppers with elephant trunks of varying lengths (Figure 177). Concrete for the 2-ft cap on the top of the lock walls was discharged directly into the form. Internal vibrators were

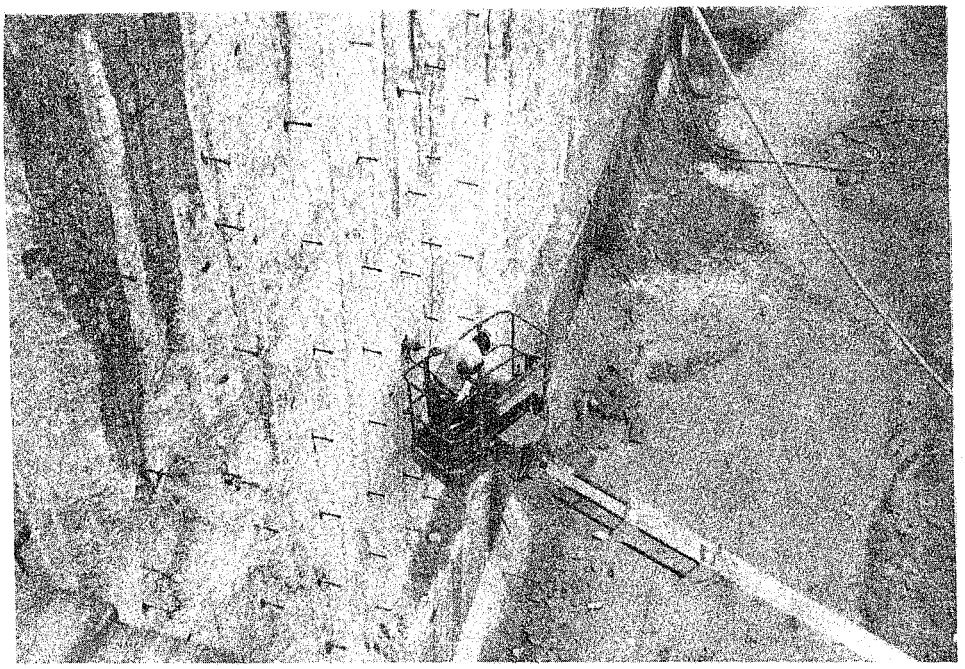


Figure 173. Load testing selected dowels, Brandon Road Lock

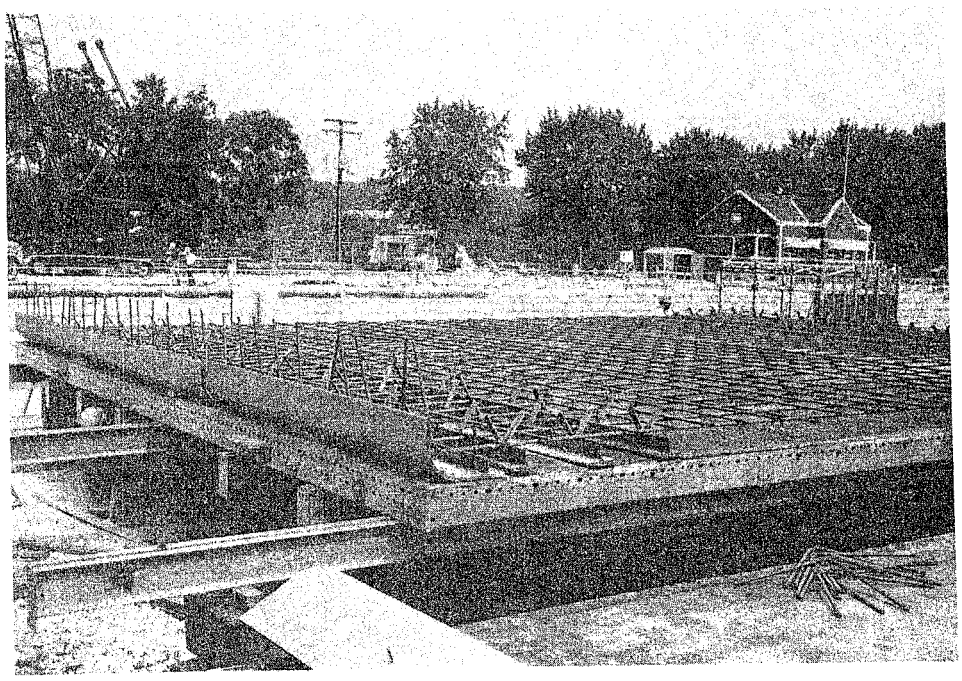


Figure 174. Wall armor and concrete reinforcement in place on the form, Brandon Road Lock

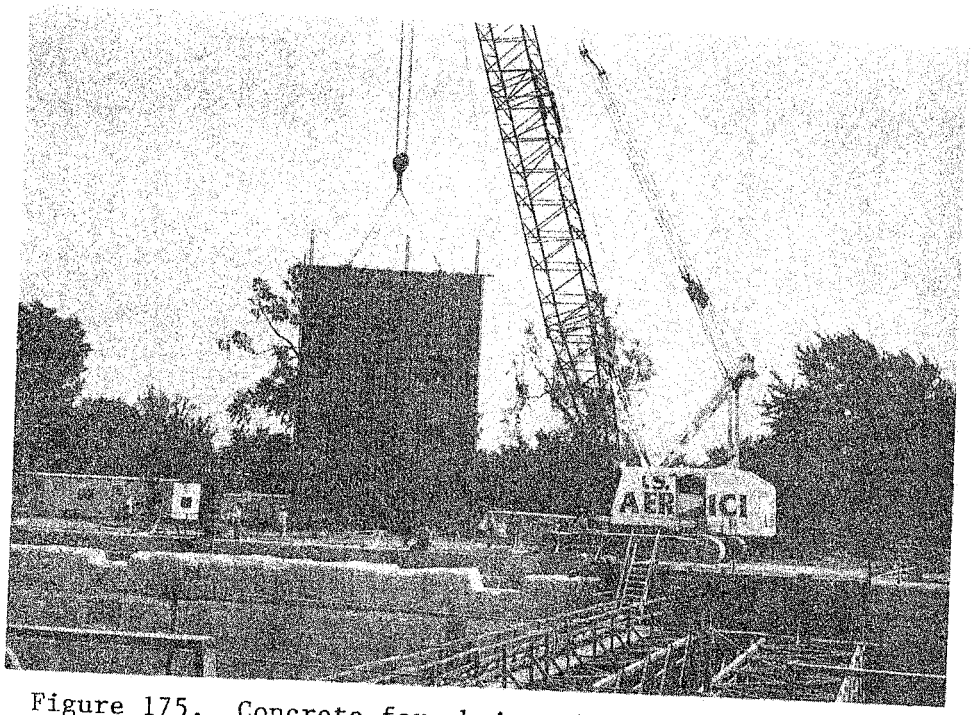


Figure 175. Concrete form being placed in position on the lock wall, Brandon Road Lock

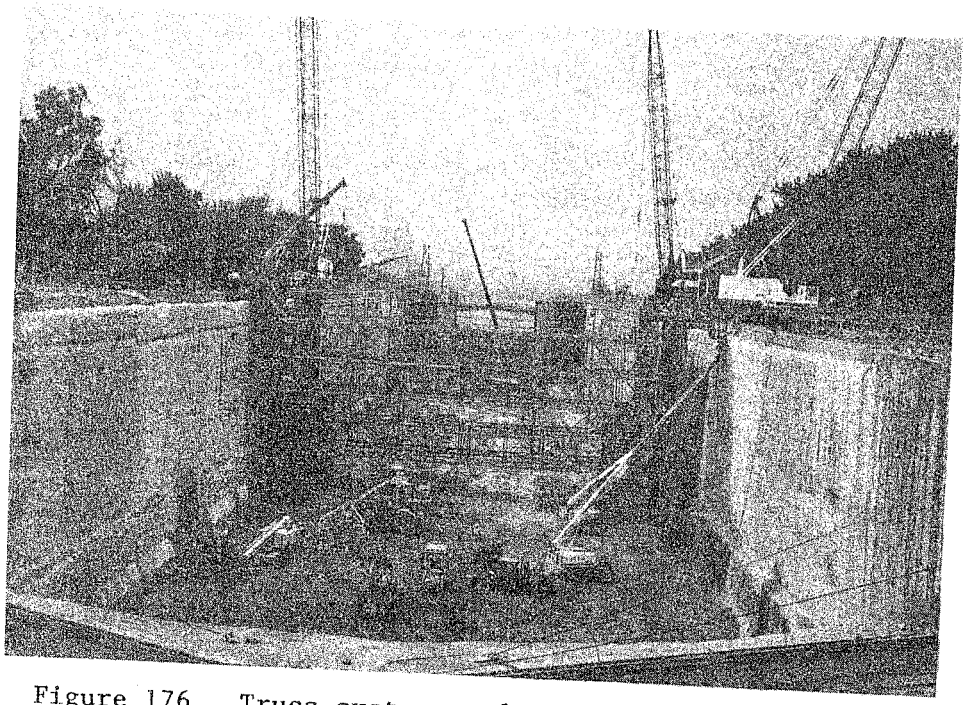
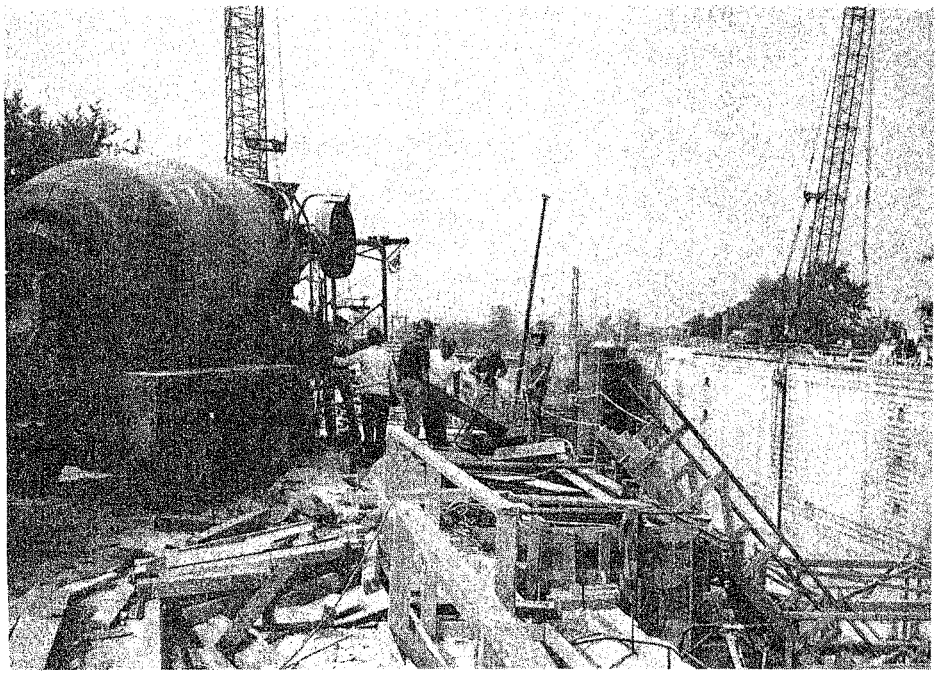
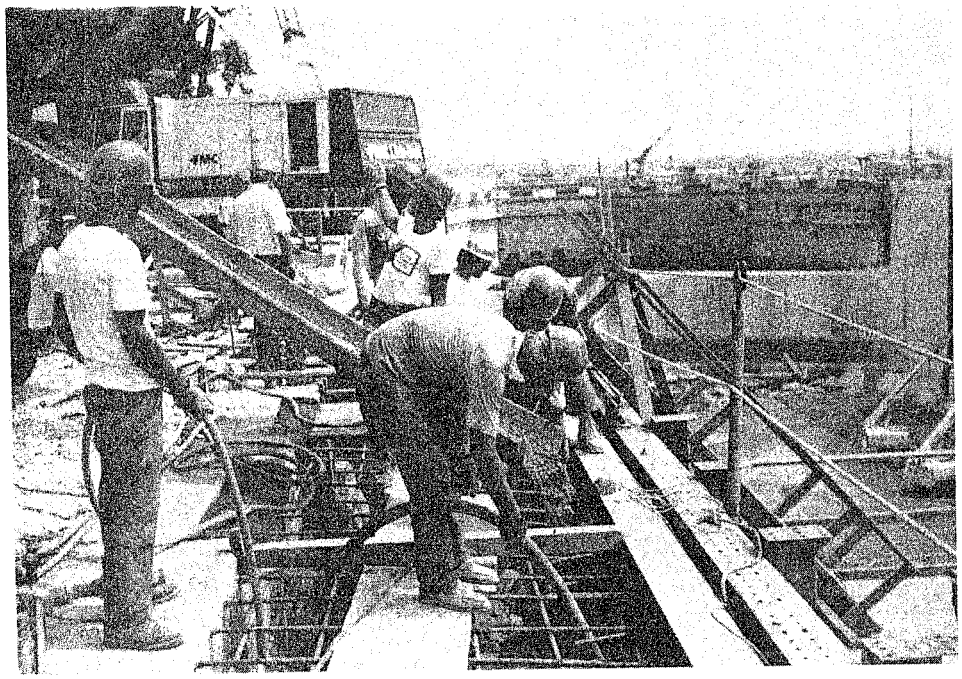


Figure 176. Truss system used to hold forms in place, Brandon Road Lock



a. Vertical face of lock wall



b. Top of lock wall

Figure 177. Concrete-placing operations, Brandon Road Lock

used to consolidate the fresh concrete. Because of the limited space and the depth, there was some difficulty in vibrating the concrete near the bottom of the lift in a few monoliths. The perimeter of these isolated areas was saw cut and the concrete chipped to a minimum depth of 2 in. prior to repair (Figure 178).

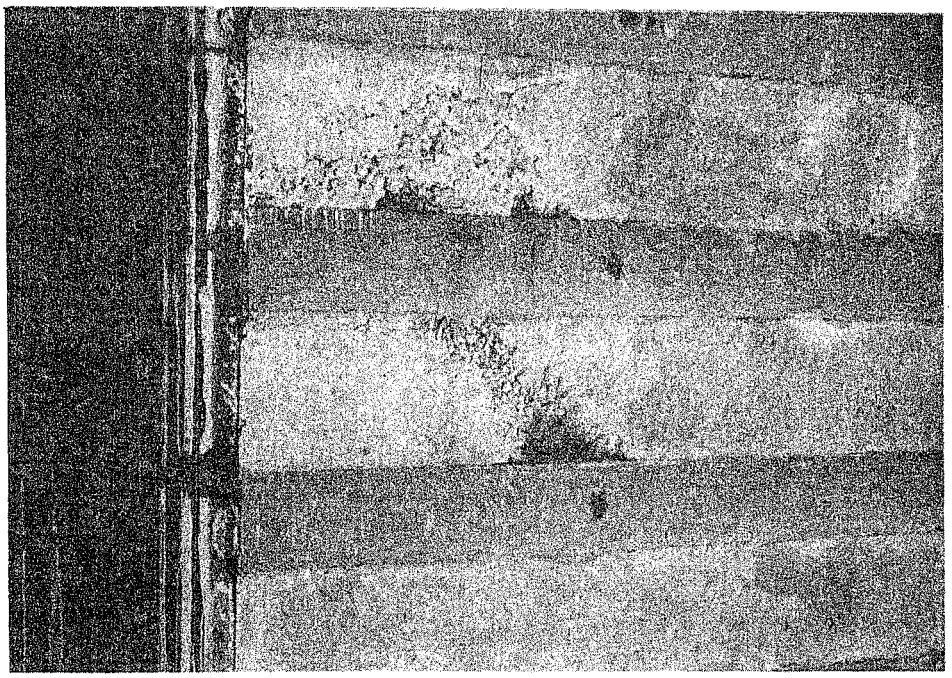
339. Maximum allowable temperature of the concrete prior to placing was 88°F, and the actual placing temperature was approximately 85°F. Ambient temperature during most of the concrete placements was in the 80's. Forms were generally stripped one day after the concrete was placed, and a membrane curing compound was applied to the formed concrete surfaces. At this point, cracking was observed in a number of monoliths (Figures 179 and 180). These cracks were generally horizontal at a spacing of roughly 5 ft. According to Corps project personnel, there appeared to be more cracking on the shaded side of the lock chamber (riverside wall) as compared to the landside wall which gets direct sunlight. This difference in cracking might be expected since the thermal gradient across the replacement concrete during cooling would be greatest for the shaded wall.

340. In an effort to obtain a nonskid surface on top of the lock walls, an abrasive material was broadcast on the concrete surface just prior to final finishing. This material was later exposed by lightly sandblasting the concrete surface. Approximately 2,800 cu yd of Type C concrete was required for resurfacing the lock chamber. The contractor's bid price for this concrete was $230 per cu yd. In addition, approximately 850 cu yd of Type D concrete was used, primarily to fill an existing gallery (Figure 163) prior to the resurfacing. The Type D concrete was proportioned for a slump of 1 to 4 in., an air content of 6 ± 1-1/2 percent, and 3,000-psi compressive strength at 28-days age. Concrete mixture proportions for a 1-cu yd batch were as follows:

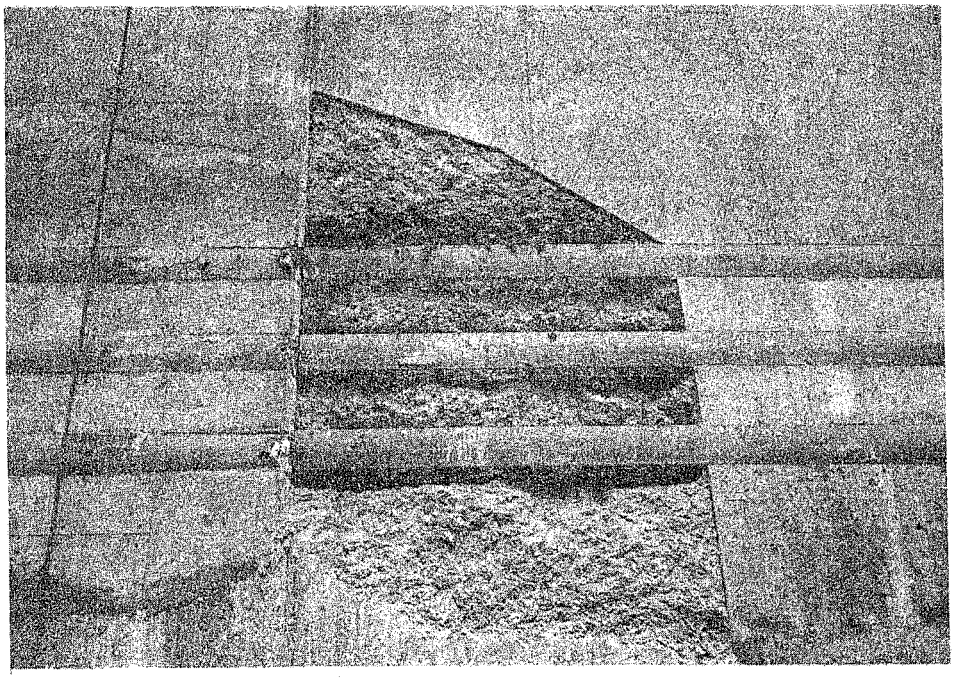
Type I cement, lb	470
Natural sand, lb	1,420
Limestone coarse aggregate, lb	1,775
Water, gal	28
Air-entraining admixture, oz	5.0

The contractor's bid price for this concrete was $120 per cu yd.

341. Resurfacing of the upper and lower gate bays and the lower gate forebay required removal of approximately 1,000 cu yd of concrete. Using explosives to remove concrete near machinery recesses and gate anchorages in

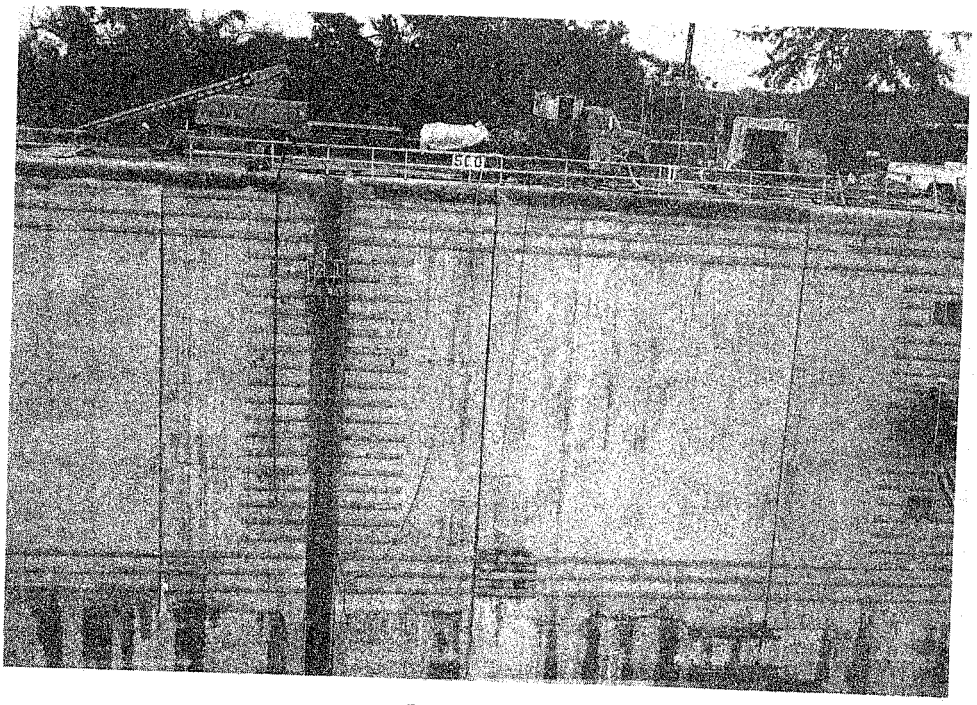


a. Concrete honeycomb

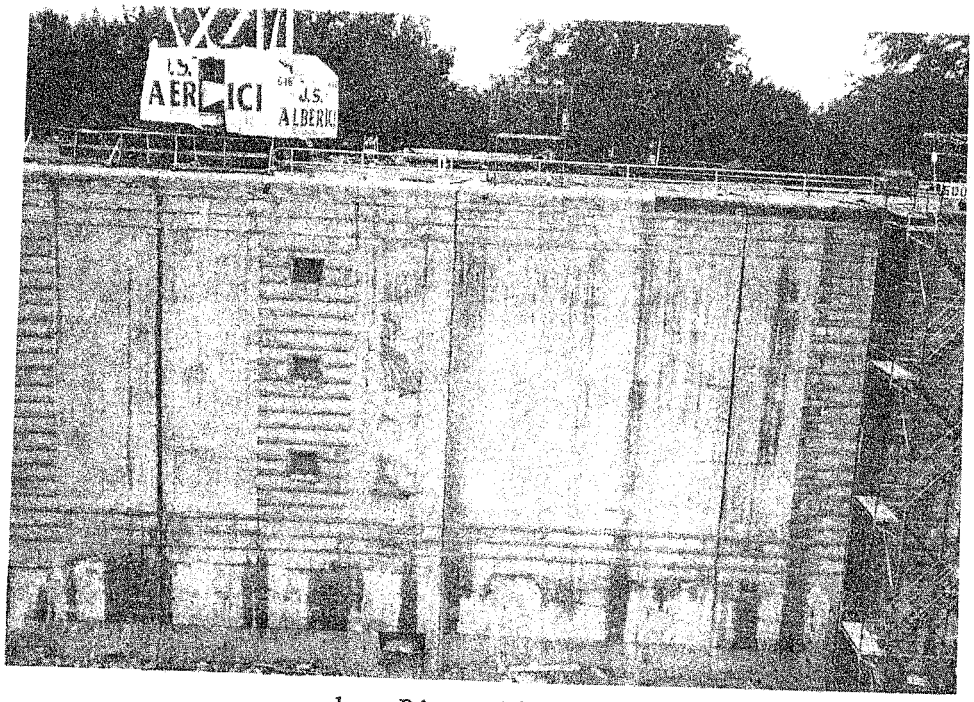


b. Area of concrete honeycomb prepared for repair

Figure 178. Repair of concrete honeycomb, Brandon Road Lock



a. Landside wall



b. Riverside wall

Figure 179. Typical cracking in the replacement concrete August 1984, Brandon Road Lock

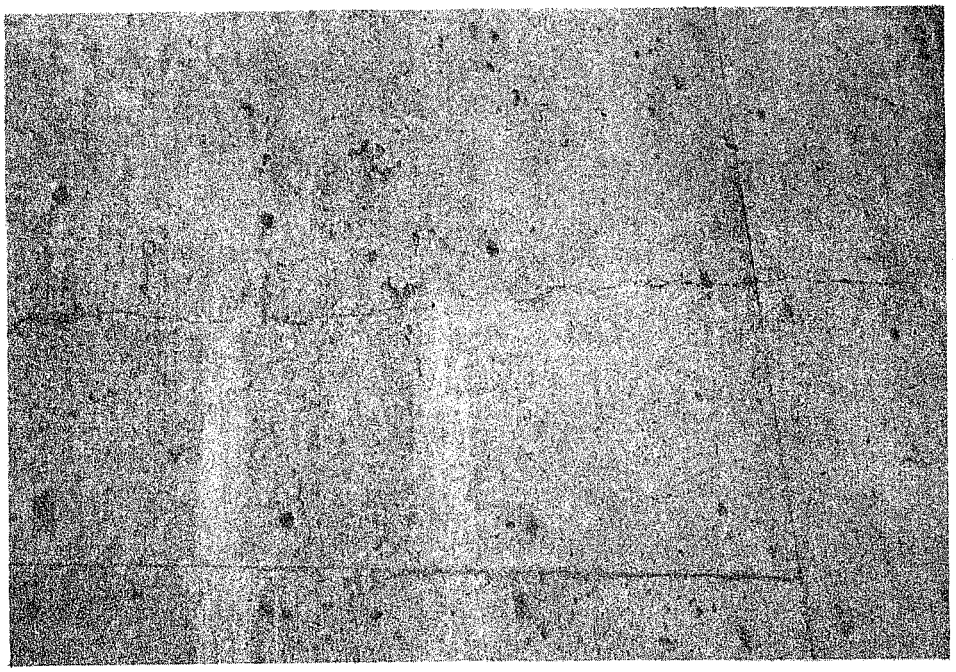
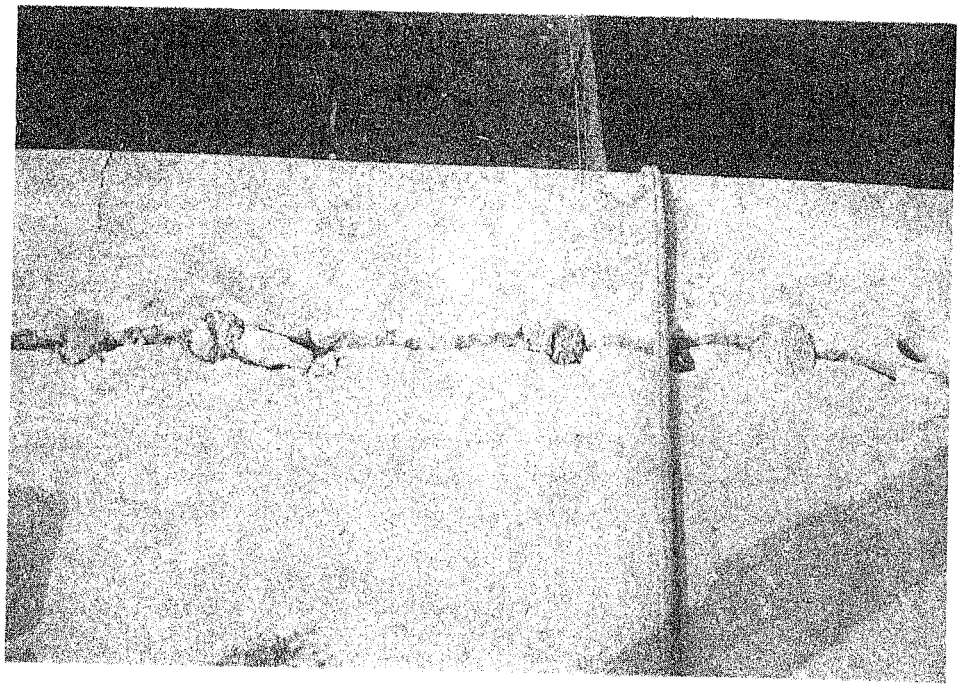


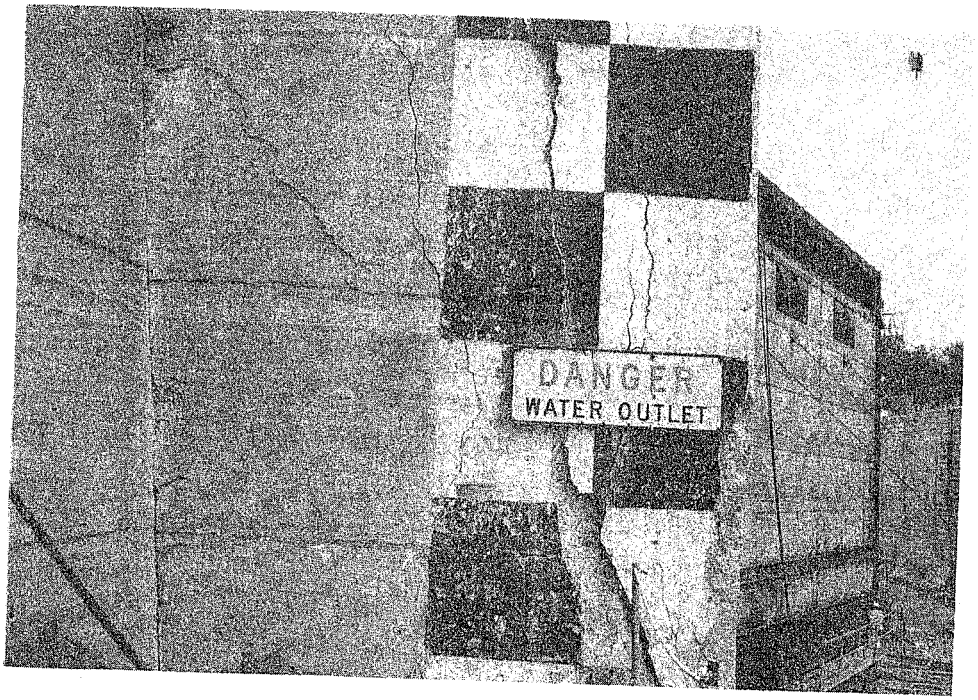
Figure 180. Typical crack in replacement concrete, August 1984, Brandon Road Lock

these areas was prohibited in the original design. In these areas, the removal line was drilled much the same as for blasting, and the contractor elected to use expansive grout (S-Mite) to fracture the deteriorated concrete face (Figure 181). A crack developed behind the quoin anchorage similar to what had happened at Lockport Lock. Initially, the fractured concrete was removed with handheld and machine-mounted breakers (Figure 182). This method proved to be very time-consuming. In an effort to increase production, removing some of this concrete with explosives was allowed. In these cases, removal was limited to approximately 5-ft intervals for each blast (Figure 183) as compared to full monolith face removal in the lock chamber. Ultimately, the Cutter Boom proved to be extremely effective in grinding off the fractured concrete in these areas (Figure 184). The contractor's bid price for removal of concrete in these areas ranged from $216 to $675 per cu yd with an average of $413 per cu yd. This average cost was approximately 2.5 times the cost of concrete removal in the lock chamber resurfacing.

342. The crack behind the quoin anchorage area was pressure-injected with epoxy grout, and the cracked section was anchored to the main concrete mass with reinforcing steel. The entire area was then resurfaced with an overlay of anchored concrete. No visual structural problems have been noted



a. Top of lock wall



b. Vertical face

Figure 181. Examples of deteriorated concrete fractured using expansive grout, Brandon Road Lock

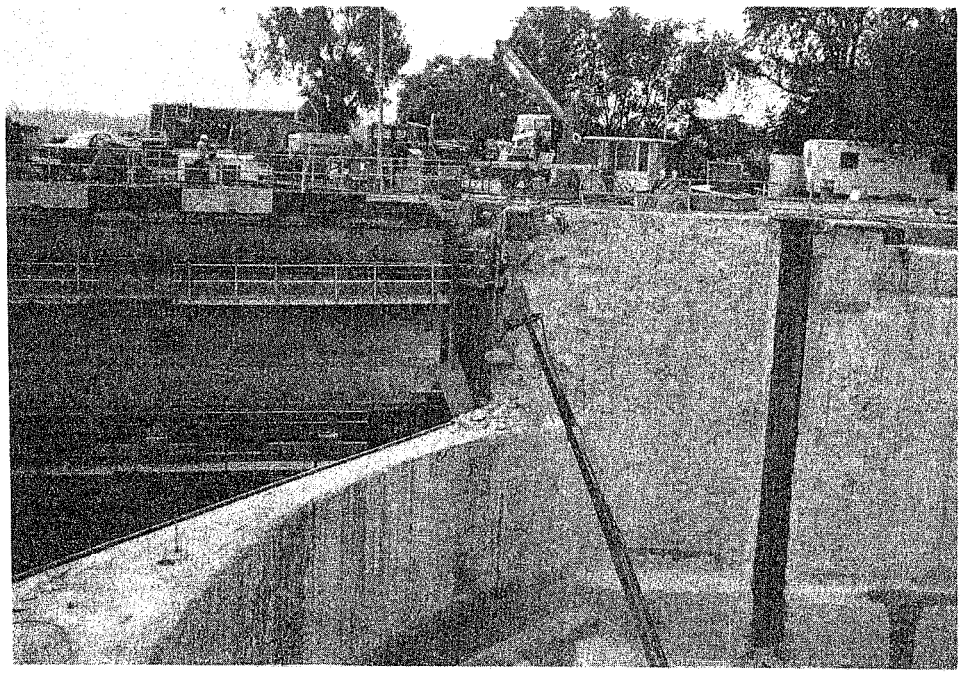


Figure 182. Concrete removal near upper miter gate, Brandon Road Lock

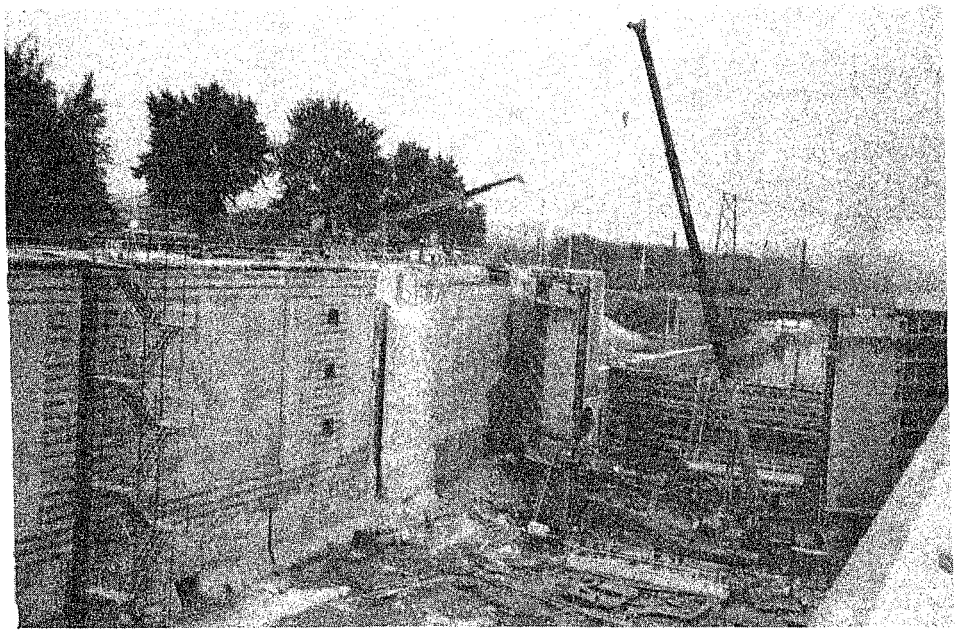
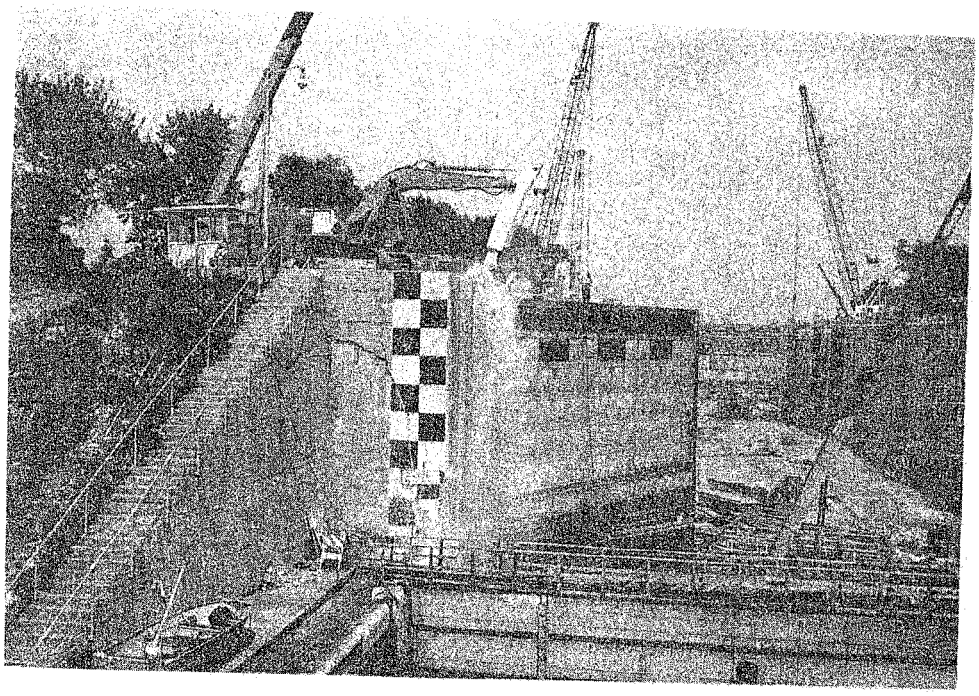
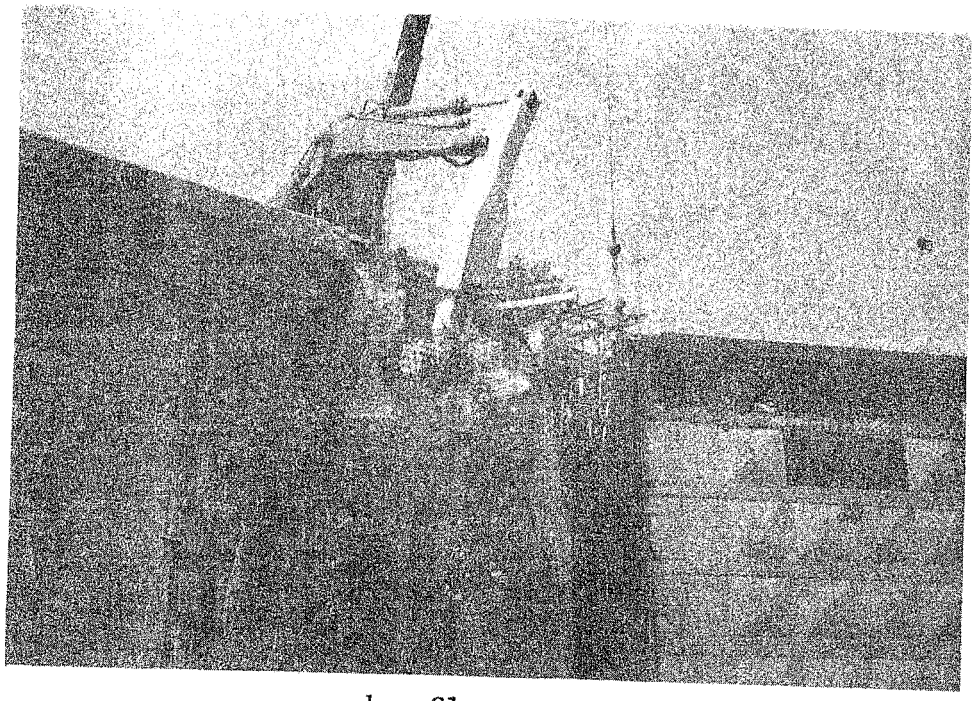


Figure 183. Blasting mat in position behind the lower miter gate for limited concrete removal, Brandon Road Lock



a. Overall view



b. Close-up

Figure 184. Cutter head removing concrete which had previously been fractured using expansive grout, Brandon Road Lock

since the repair was made (Rock Island District 1985).

343. The top 8 ft of the river face of the upper guide wall was resurfaced with a nominal 8 in. of new concrete. This work required removal of approximately 225 cu yd of existing concrete. New wall armor and horizontal corner armor were installed as shown in Figure 185. Typical views of the

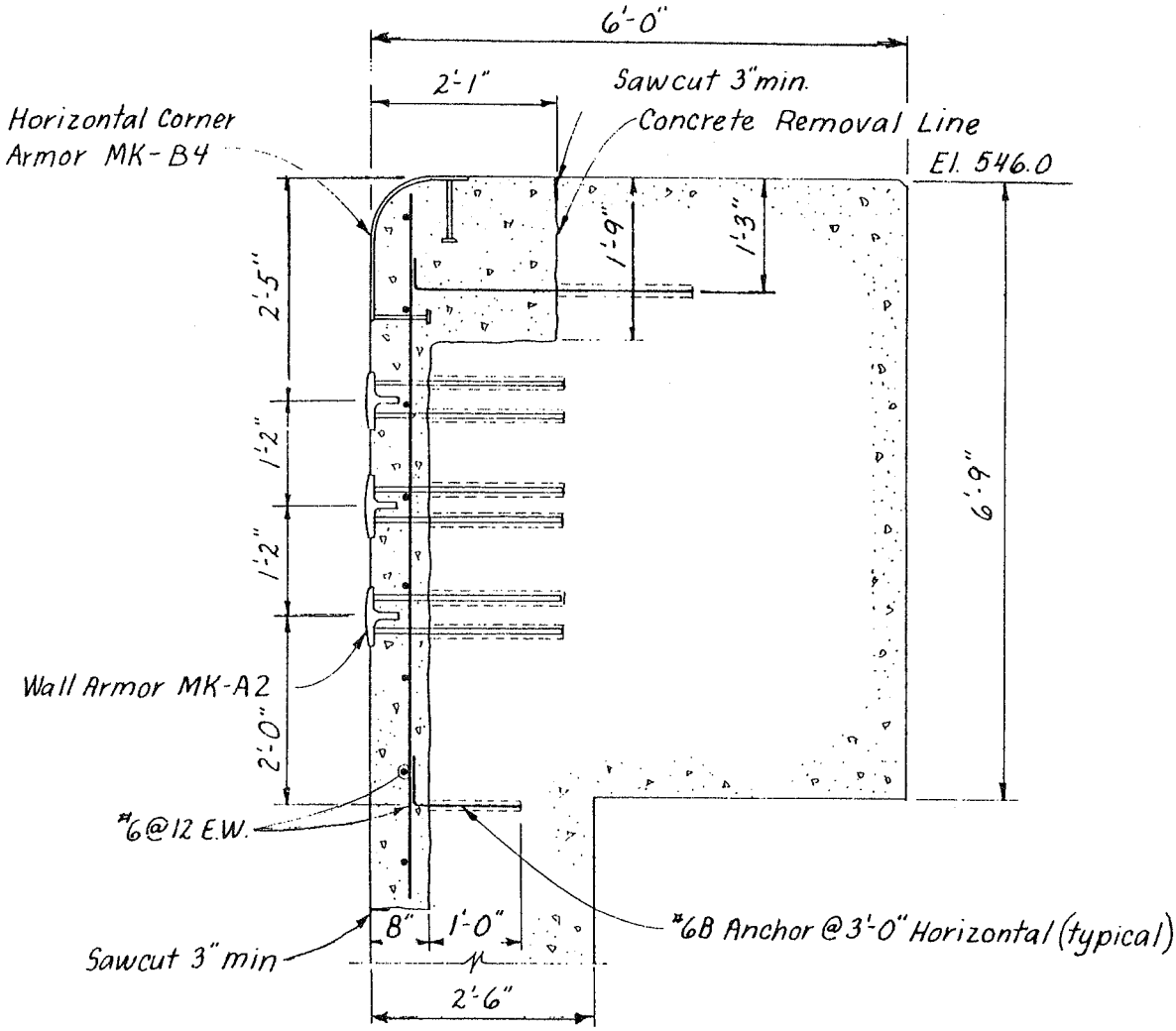
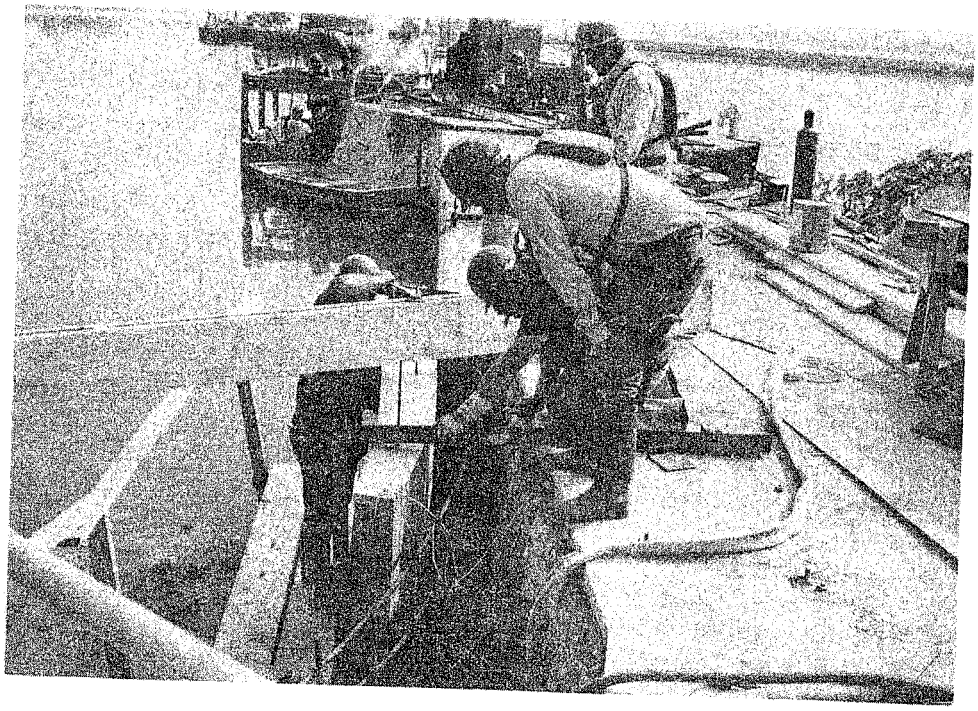
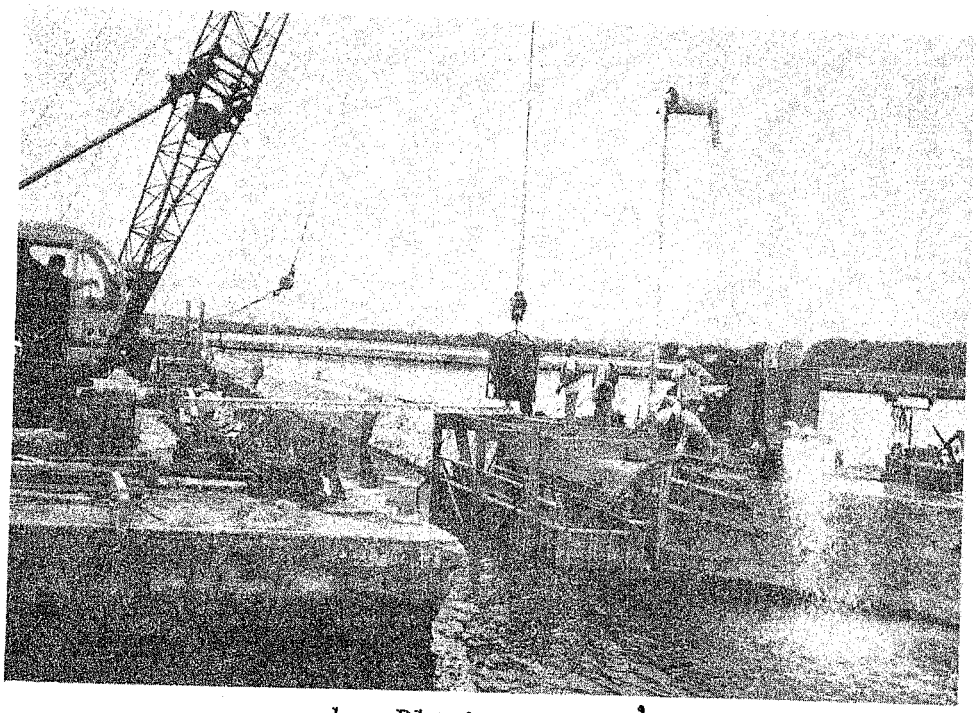


Figure 185. Upper guide wall resurfacing details, Brandon Road Lock

resurfacing operation are shown in Figure 186. Type C concrete was used to reface the upper guide wall. Approximately 2 to 3 in. of concrete was removed from the top surface of the guide wall and replaced with an overlay of latex-modified concrete (approximately 17 cu yd). Latex-modified concrete (Figure 187) was specified in an attempt to reduce the shrinkage cracking problems inherent with conventional concrete overlays. The latex additive made the concrete surface very sticky and thus caused some finishing problems.



a. Form preparation



b. Placing concrete

Figure 186. Upper guide wall and gate bay resurfacing, Brandon Road Lock

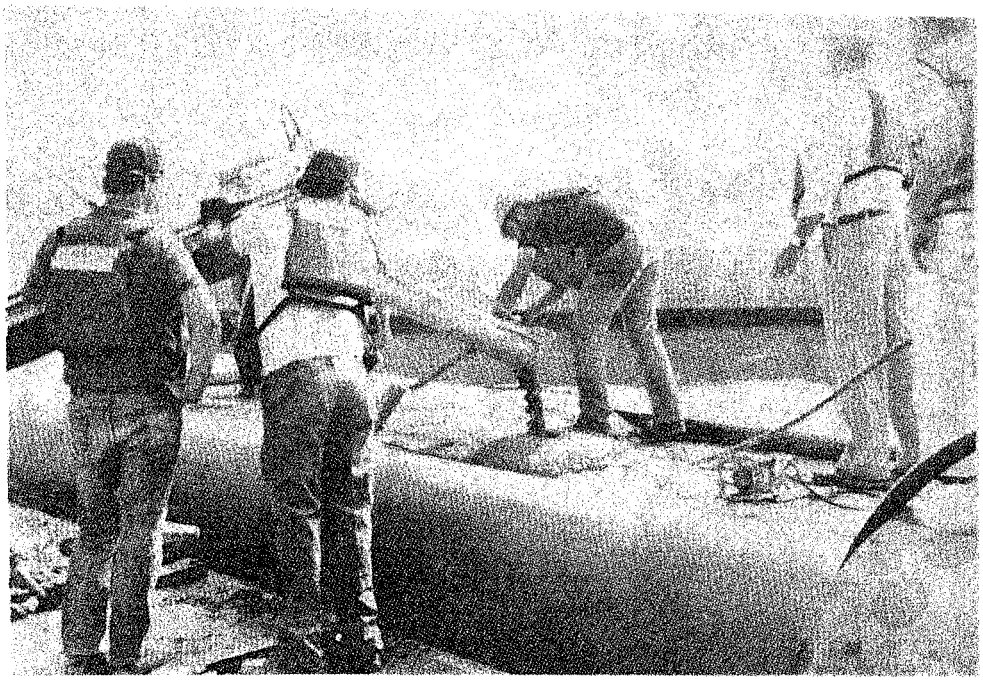


Figure 187. Resurfacing top of the upper guide wall with latex-modified concrete, Brandon Road Lock

However, the latex-modified concrete has performed well with a minimum of cracking. The contractor's bid price for latex-modified concrete was $750 per cu yd, and the total bid price for resurfacing the upper guide wall was approximately $350,000.

344. Rehabilitation of the lower guide wall required stabilization and concrete resurfacing along its entire length. The existing wall had a 3-ft top width with a 3 on 12 batter. In order to resurface the riverside face and stabilize the walls, the design called for complete removal of 10 ft off the top of the guide wall to 1 ft below waterline (Figure 188). As a result, the concrete was easier to remove, and a wider base for the required anchorage system was provided. The anchors consisted of seven 1/2-in.-diam stranded anchors on 7-ft 6-in. centers in a double corrosion protection system. The wall was rebuilt with a top width dimension of 5 ft 6 in. to improve operational safety conditions. The contractor's bid price for stabilization and resurfacing the lower guide wall was approximately $900,000.

345. Rehabilitation of the upper and lower miter gates involved replacement of corroded rivets with high-strength bolts, replacement of corroded structural members, and sandblasting and painting of the entire gates (Figure 189). Many additional members that were not identified during the

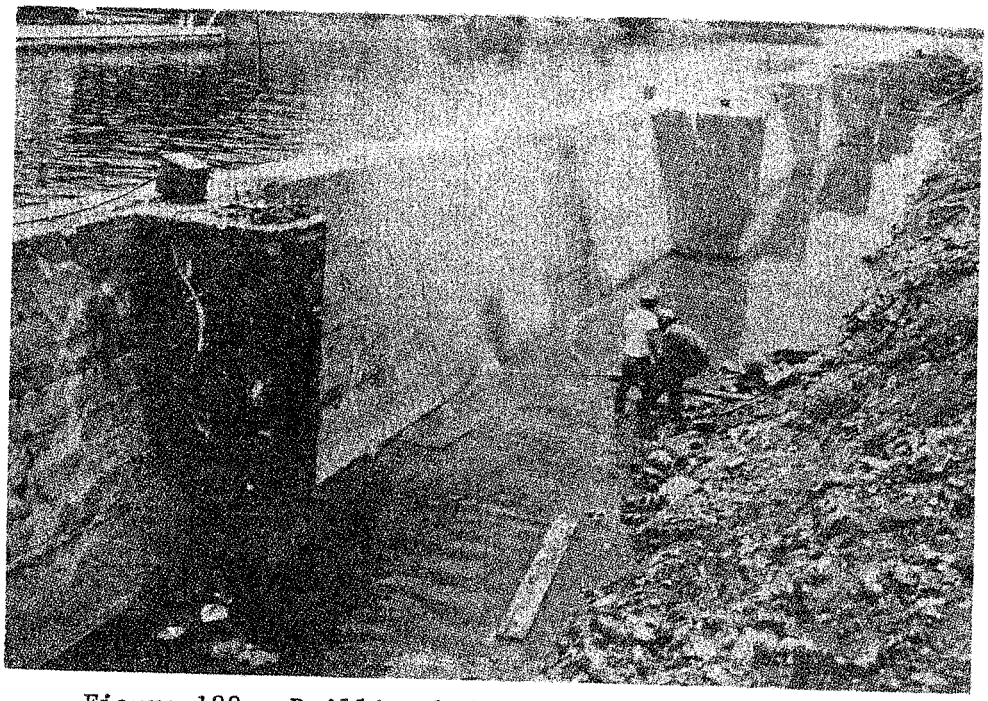
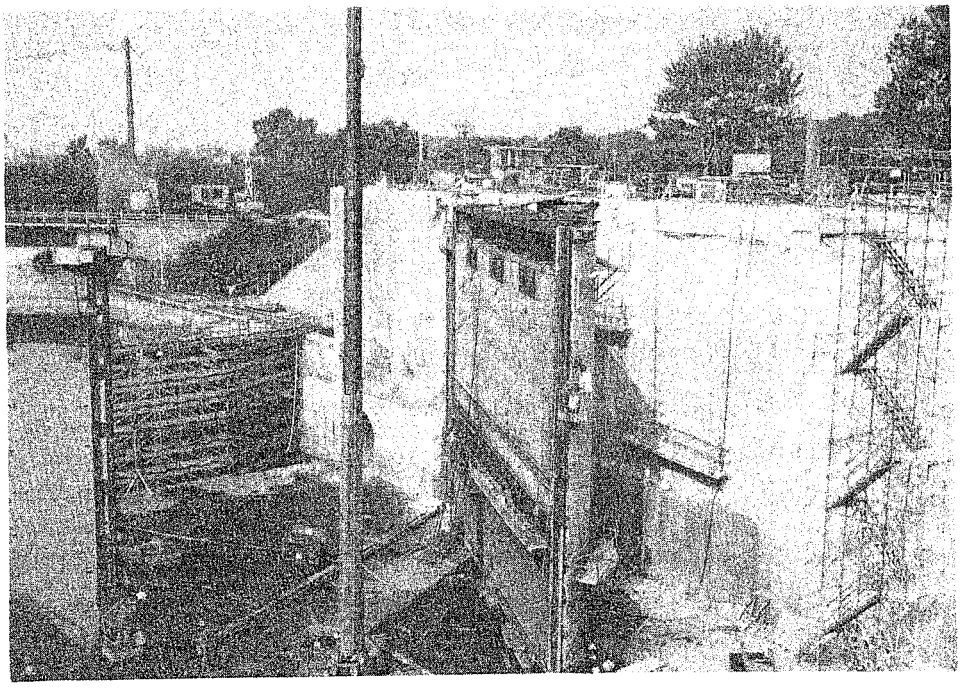


Figure 188. Drilling holes in the lower guide wall prior to removal, Brandon Road Lock

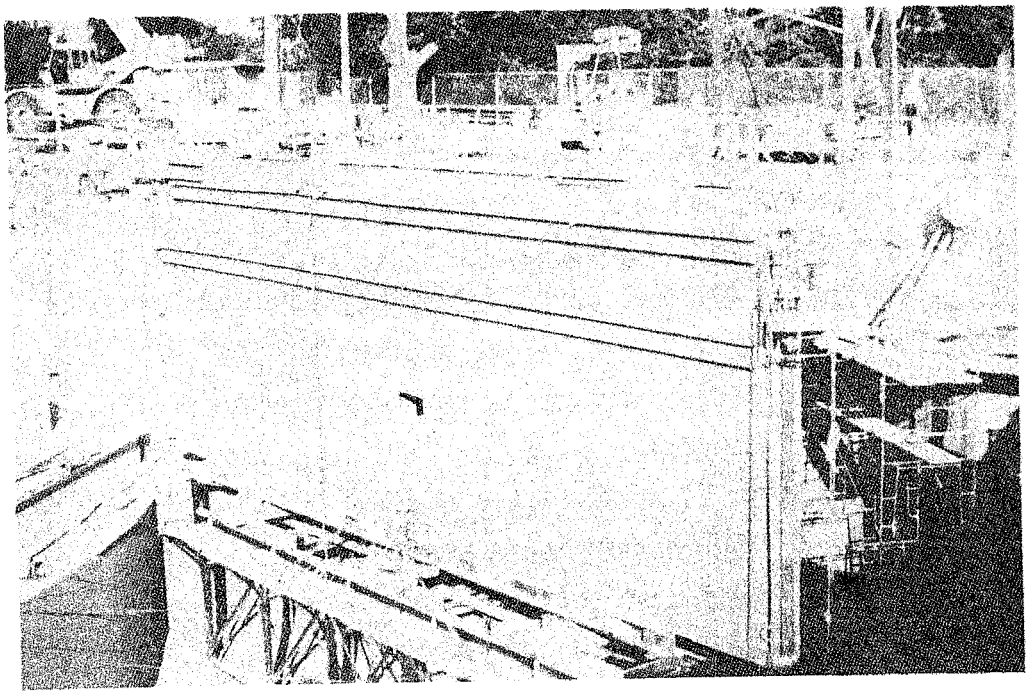
original inspection were also replaced. The gates were moved off their anchorages and set adjacent to their original location. Delays were encountered during final placement and adjustment of the lower miter gates. The contractor had no experience in miter gate work, and progress was very slow. Consequently, the Corps of Engineers provided assistance with its own structural maintenance personnel to help complete final placement and adjustment of the lower miter gates.

346. The contract required complete replacement of the electrical distribution system prior to the lock closure. Thus, the cables and conduit had to be relocated out of the gallery into a new cable raceway. All work could be completed prior to the lock closure except for the electrical crossover located in the bottom of the lock. This approach was taken to minimize start-up problems after the lock was reopened to navigation on 29 September 1984, within hours of the reopening of Lockport Lock.

347. Currently, rehabilitation work on the Brandon Road Dam and on the walls upstream from the dam within the city of Joliet is progressing. This work consists of mechanization of tainter gates, resurfacing of concrete, and repairs to the head gates included in a part of the dam originally intended for hydropower. Extensive rehabilitation work on the Marseilles Dam a few



a. Lower service gate being rehabilitated



b. Rehabilitated upstream miter gate ready for installation

Figure 189. Lock gate rehabilitation, Brandon Road Lock

miles downstream is also in progress. Contracts for rehabilitation of Peoria and LaGrange Locks and Dams were awarded in 1986. Rehabilitation of Locks and Dams 11 through 22 on the Mississippi River will follow.

348. Most of the rehabilitation work at Brandon Road and Lockport Locks was accomplished during a 90-day period when the waterway was closed to navigation. Rehabilitation of each lock by a different contractor during the same time period led to a unique opportunity to evaluate construction techniques and construction management technology, scheduling, and quality control. A report to document the rehabilitation of the two locks and to describe the results of various methods used for concrete rehabilitation (such as removal and replacement, epoxy injection of cracks, latex-modified concrete overlay, and resin cartridge anchors), miter gate replacement, miter gate repair, and installation of posttensioned anchors in bedrock was prepared by the Rock Island District (1985). Some of the lessons learned are already being applied to ongoing rehabilitation work at these and other sites on the Illinois Waterway and for work being planned for the Upper Mississippi River system of locks and dams about to get underway. It is expected that some of the lessons learned on this project will serve as a guide to similar work throughout the nation's waterways. Also, this report points out some areas where further research is needed.

Lessons learned

349. The completion of nearly $16,000,000 worth of complicated rehabilitation work in an 86-day period was a major success. The fact that it had been scheduled for 65 days reflects the inability of the District to foresee many problems that would inevitably come up during the construction of a major project of this nature. One of the major lessons learned is that scheduling should provide ample time for the work to be completed where major transportation systems are so vitally affected. A 90-day schedule of work accomplished in 86 days would have been vastly better than the 65-day schedule accomplished in 86 days. A list of lessons learned is included below, representing a composite gathered from management, engineering, operations, and construction personnel of the District:

a. Determine realistic schedules for lock closures. Allow enough float in the completion date to take into account problems with weather, changed site conditions, and labor problems.

b. Provide a model CPM for the contractor to use a guide. Lay out a sample job for the contractor to give him less opportunity to fail.

c. Detail Design and Operations personnel to the field to help interpret design and operational requirements and to expedite solutions to changed conditions if and when encountered.

d. Ensure rapid communications between construction and engineering offices. Guarantee a quick response on shop drawing submittals and design changes.

e. Consider some means, or devise a way, to reduce the contract payments in cases where some contract work is done by Corps personnel to expedite completion.

f. Require a full-time quality control coordinator in special circumstances. Under tight time schedules, allowing the contractor's personnel to have double duties in quality control is not a good idea. It is suggested that making quality control a bid item would provide greater incentives in this area.

g. Show details on the plans rather than in the specifications. Contractors tend to use the plans to build and the specifications for legal purposes.

h. On important work with tight schedules, there should be a way to prequalify contractors and subcontractors to ensure experience.

i. Incentives need to be devised to give contractors added reasons to finish jobs on time. The present system of liquidated damages does not and is not intended to give our contractors incentives for early completion, or even on the original date set, if excusable delays are encountered.

j. Do not use expansive grout for concrete removal unless it is completely contained in an impervious liner.

k. Consider using wall armor only at critical locations on lock walls.

l. Expect cracks in the quoin areas of lock walls to be generally more extensive than indicated on the surface. These are areas of high impact and high service load.

m. Consider the use of district operations personnel to rehabilitate miter gates. Generally, contractors lack experience to do this work and often do not have the equipment needed. Also, many unforeseen repairs make fixed-price contracts very awkward to administer.

n. Removal and replacement of deteriorated concrete may only be cosmetic and in many cases the appearance may not be satisfactory either. Damage from freezing and thawing may only be a few inches deep. Replacement concrete can be expected to crack extensively, even though it is reinforced and anchored.

o. Latex-modified concrete placed on horizontal surfaces, well prepared in advance by grinding, chipping, or sandblasting,

using styrene butadiene latex and following the manufacturer's recommendations, seems to be a proven, successful system. Other latex and applications thinner than 1-3/4 in. should be tested under job conditions.

p. Concrete anchors set in cartridges of resin grout have few problems. The problem of installing hooked anchors can be solved by replacing them with a "Richmond" type anchor, by using a counter balanced drill to spin the hooked bar, or by mixing the resin with a smaller straight rebar and replacing it before set time with a shoved in hooked bar. Additional tests should be done on the last method to assure anchorage strength is not compromised.

q. Epoxy injection of cracks in concrete is a fairly common repair but there is no guarantee of results.

r. Some bonding agents applied on concrete that will be placed in service below water may do more harm than good.

s. High-strength steel bolts used to replace rivets on water-retaining steel structures generally leak. Seal welding may be needed where rusting can occur between bolted or riveted parts.

t. Use of epoxy as a filler in quoin and miter block areas should be reexamined. The use of babbitt as an alternative should be provided for and may be necessary during some temperature conditions.

There was one rather unique problem or perception of a problem on the part of the contractor at Lockport that may be put under "lessons learned." The employment situation in the Joliet area was somewhat depressed, and the contractor faced some resistance by labor to putting forth extra effort to complete early, or within the time scheduled, for the reason that when the job was done there was no other work to be had. The Brandon Road contractor did not have this problem to the same degree since his contract continued with other work beyond the closure period.

Recommendations for further research

350. The following list of recommendations covers areas that the District felt needed to be addressed in more detail by the Corps of Engineers. Such research would help the Corps determine if the useful life of existing structures can be extended effectively by implementing these methods.

a. Methods for removing unsound concrete at the surface and the ability to control the removal. Examples of removal techniques are high-pressure water-jet blast (Conjet), steel shot, grinding, blasting and the use of expansive grouts contained in impervious liners.

b. Replacement of removed concrete by shotcrete applications instead of traditional cast-in-place alternatives.

c. Replacement of removed concrete by installing precast wall panels.

d. Thin resurfacing techniques on concrete horizontal surface such as sealants, epoxies, and trowel on finishes.

e. Pull test results on methods of installing hooked bars into existing concrete by premixing the two-component adhesive with a smaller diameter straight bar.

f. Using various epoxy injection materials to act as bonding agents between separated masses.

g. Methods of field bolting to accomplish watertight joints on fabricated items such as miter gates and tainter gates.

h. The use of epoxy fillers for high compressive stresses.

i. Using stressed versus unstressed anchors in stabilizing concrete structures founded on rock.

j. Methods to limit shrinkage cracking in cast-in-place concrete repairs.

PART III: DISCUSSION

351. Approximately half of the Corps 269 navigation lock chambers were built prior to 1940. Consequently, the concrete in these structures does not contain intentionally entrained air and is therefore susceptible to deterioration by freezing and thawing. Since more than three-fourths of these older structures are located in the Corps North Central and Ohio River Divisions, areas of relatively severe climatic exposure, it is not surprising that the concrete in many of these structures exhibits significant freeze-thaw deterioration. Depending upon exposure conditions, depths of concrete deterioration can range from surface scaling to several feet.

352. The general approach in lock wall rehabilitation has been to remove the deteriorated concrete and replace it with concrete or shotcrete. Explosive blasting has been used successfully at several Corps projects and appears to be the most cost-effective and expedient means for removing large quantities of concrete. In order to minimize damage to the concrete that remains after removal, controlled blasting techniques have been developed based on test blasts or results of previous work. This procedure generally involves drilling a line of small boreholes parallel to the removal face, loading each hole with light charges of explosive (usually detonating cord), cushioning the charges by stemming the hole, and detonating the explosive with electric blasting caps. The selection of proper borehole spacing and charge weight depends upon the location of the structure or element, acceptable degree of vibration and damage, and the quantity and quality of concrete to be removed.

353. Following blasting, it is usually necessary to scale lock walls to remove loose concrete still clinging to the wall surfaces. The amount of scaling required is usually proportional to the amount of reinforcing, normally form tie steel, present in the concrete. This scaling, normally done by manual processes such as labor crews working from scissor lifts or scaffolding with handheld hammers and chipping tools, is a very costly and time-consuming operation. The Cutter Boom, a modified piece of mining equipment having a rotary head cutter with a series of carbide-tipped teeth that grinds away the concrete, was effectively used to scale the walls at Brandon Road Lock. In addition to the scaling operation, deeper cuts were easily made by the Cutter Boom to accommodate lock chamber appurtenances such as exit ladders, line hooks, and snub posts that required localized deeper embedment into the

existing structure than was afforded by the standard removal line.

354. Concrete removal by blasting has been generally prohibited in sensitive areas such as gate monoliths for fear of damage to the remaining concrete around gate anchorages. Commercially available expansive agents have been used to presplit the concrete in such areas with mixed results. The expansive agents, usually in slurry form, are placed in boreholes drilled at points along a predetermined line to induce a crack plane which allows for removal of the concrete. However, if the removal line is not located within sound concrete, or the concrete mass contains fractures which allow the slurry to penetrate the mass, very erratic crack patterns may result. Impervious membranes can be used as borehole liners to maintain the slurry within the borehole. In an effort to increase production, some limited blasting was allowed to remove concrete from the gate monoliths at Brandon Road Lock. In this case, removal was limited to approximately 5-ft intervals for each blast with no apparent damage to the remaining concrete. The advantages and limitations of a variety of concrete removal techniques are described in EM 1110-2-2002 (HQUSACE 1986).

355. Once the deteriorated concrete has been removed, conventional cast-in-place concrete has been used as the replacement material in most lock wall rehabilitation projects. Other replacement systems that have been used or proposed include shotcrete, preplaced-aggregate concrete, and precast concrete stay-in-place forms. In addition, several materials including latex-modified concrete, polymer mortars and grouts, conventional shotcrete, and latex-modified, fiber-reinforced shotcrete have been used as thin overlays on existing lock walls.

Conventional Concrete Replacement

356. The economics of conventional forming and concrete replacement compared to other rehabilitation techniques usually depends on the thickness of the concrete section to be replaced. For sections in the range of 6 to 12 in., both formed and nonformed techniques such as shotcrete are economically competitive. When the thickness of the replacement section exceeds 12 in., conventional form work and concrete replacement are generally more economical. Conventional concrete replacement has several advantages over other rehabilitation techniques including: (a) replacement concrete mixtures

can be proportioned to simulate the existing concrete substrate, thus minimizing strains caused by material incompatibility; (b) proper air entrainment in the replacement concrete can be obtained by use of admixtures to ensure resistance to cycles of freezing and thawing; and (c) materials, equipment, and personnel with experience in conventional concrete application are readily available in most areas.

357. Once concrete removal is completed, wall surfaces are usually washed with high-pressure water jets to clean the surface of any materials that could inhibit bond. Dowels are normally used to anchor the new concrete facing to the existing concrete walls and to position vertical and horizontal reinforcing steel in the concrete facing. In most of the earlier rehabilitation work, dowels were arbitrarily spaced 2 ft on centers each way. Based on laboratory and field tests (Liu and Holland 1981), the current practice is to use No. 6 deformed reinforcing bars spaced 4 ft on centers each way except in the vicinity of local openings and recesses, and along the perimeter of monoliths where 2-ft spacings may be specified. Both polyester resin and cementitious grouts have been used to embed anchors in holes drilled with rotary-percussion equipment. Prepackaged polyester resin grout has been used on most of the recent projects and field pullout tests of replacement concrete anchors installed under dry conditions indicate this procedure to be satisfactory. However, polyester resin grouting of posttensioned anchors under wet conditions resulted in a number of anchor failures at Lock No. 3, Monongahela River. Subsequent laboratory tests (McDonald and Best 1987) indicate the pullout strength of anchors grouted with polyester resin under submerged conditions is one-third to one-half less than the strength of similar anchors grouted under dry conditions.

358. Mats of reinforcing steel, usually No. 5 or No. 6 bars on 12-in. centers each way, are hung vertically on the dowels. Selection of bar sizes and spacing appears to be based on EM 1110-2-2103 (HQUSACE 1971) which requires, for members restrained at one face, a reinforcement of 0.2 percent of the gross cross-sectional area, half in each direction near the unrestrained face, with a maximum of No. 6 bars at 12 in. Concrete cover over the reinforcing is usually a minimum of 4 in. In some cases, the reinforcing mat, wall armor, and other lock wall appurtenances are installed on the form prior to its being positioned on the face of the lock wall.

359. Once the reinforcement and form work are in position, replacement

concrete is usually placed by pumping or discharging it directly into hoppers fitted with various lengths of flexible pipe commonly known as elephant trunks. Lift heights varying from 5 ft to full face of approximately 40 ft have been used. Normally, concrete is placed on alternating monoliths along a lock chamber wall. Generally, forms are removed one day following concrete placement, and a membrane curing compound applied to formed concrete surfaces.

360. One of the most persistent problems in lock wall rehabilitation with the use of this approach is cracking in the replacement concrete. In all of the Corps rehabilitation projects to date, the resurfacing concrete exhibits some degree of concrete cracking. These cracks, which in some cases extend completely through the replacement concrete, are attributed primarily to the restraint provided through bond to the stable mass of existing concrete. As the relatively thin layer of resurfacing concrete attempts to contract as a result of plastic and drying shrinkage, thermal gradients, and autogenous volume changes, tensile strains develop in the replacement concrete. When these strains exceed the ultimate tensile strain capacity of the replacement concrete, cracks develop. In most cases, such cracking will not cause structural deficiencies. However, the cracks are unsightly and may require additional maintenance to minimize deterioration.

361. Concrete materials, mixture proportions, and construction procedures have been varied in attempts to minimize cracking in resurfacing concrete. The use of shrinkage compensating cement and fly ash as a partial cement replacement failed, in limited tests, at Lock No. 1, Mississippi River, to eliminate the cracking problem. Also, attempts to enforce the specified temperature differential between the concrete surface and at 2-in. depth of not more than 25°F still resulted in cracks. Horizontal cracking was controlled through the use of horizontal drummy joints installed on 5-ft centers. The replacement concrete at Old Lock No. 14, Mississippi River, containing fly ash as a partial cement replacement (approximately 20 percent by weight), exhibits significant cracking. Individual lifts of 5 ft and 40 ft of the same concrete mixture at Lockport and Brandon Road Locks, respectively, both exhibited cracking. The replacement concrete at Lock No. 3, Monongahela River, appears to exhibit less cracking than any of the projects described herein. Several factors may have contributed to this resurfacing being relatively crack free including: (a) the use of a floating batch plant which could be moved to the areas of placement, thus minimizing the time between concrete

mixing and placing; (b) 1-1/2-in. maximum-size aggregate; and (c) a combination of membrane curing compound and wet burlap for concrete curing.

362. After about 18 years in service, 6-in. of the original concrete on top of the landside wall and the downstream, riverside gate monolith at Old Lock No. 14, Mississippi River, was removed and replaced with new concrete. This resurfacing was done in about 1940, so the replacement concrete probably did not contain intentionally entrained air. About 37 years after resurfacing, vertical cores showed the 6-in. concrete cap on top of the landside wall to be in good condition; however the concrete beneath the cap was deteriorated to a depth of about 12 in. The downstream gate monolith on the riverside wall exhibited a similar condition. In comparison, the concrete on top of the riverside wall which was not capped was deteriorated to a depth of about 10 in.

363. During the original construction of Dresden Island Lock, horizontal surfaces were finished without any slope. As a result, water ponding on these surfaces contributed to freeze-thaw deterioration of the nonair-entrained concrete. After 28 years in service, the top surfaces and the upper 3 ft of the vertical walls of the lock were resurfaced by removing a minimum of 4 in. of concrete and replacing it with new air-entrained concrete reinforced with wire mesh. Eleven years later, random cracking of the resurfaced concrete on top of the lock walls was reported. This cracking was attributed in part to reflective cracking from the original concrete. After 15 years in service, four cores were obtained from the concrete resurfacing. The replacement concrete was found to be structurally sound by itself, but in certain locations it was considered susceptible to barge impact because of the frost-damaged concrete beneath.

Shotcrete Resurfacing

364. For repair of sections less than 6 in. thick, shotcrete is generally more economical than conventional concrete because of the saving in forming costs. Properly applied shotcrete is a structurally adequate and durable material, and it is capable of excellent bond with concrete and other construction materials. These favorable properties make shotcrete an appropriate selection for repair in many cases. However, there are some concerns about the use of shotcrete to rehabilitate old lock walls. The resistance of

shotcrete to cycles of freezing and thawing is generally good in some cases despite a lack of entrained air. This fact is attributed in part to the low permeability of properly proportioned and applied shotcrete which minimizes the ingress of moisture, thus preventing the shotcrete from becoming critically saturated. Consequently, if the existing nonair-entrained concrete in a lock wall behind a shotcrete repair never becomes critically saturated by moisture migration from beneath or behind the lock wall, it is likely that such a repair will be successful. However, if moisture does migrate through the lock wall and the shotcrete is unable to permit the passage of water through it to the exposed surface, it is likely that the existing concrete will be more fully saturated during future cycles of freezing and thawing. If frost penetration exceeds the thickness of the shotcrete section under these conditions, freeze-thaw deterioration of the existing nonair-entrained concrete should be expected. Under such conditions, the shotcrete becomes debonded from the existing concrete.

365. Twelve years after being resurfaced with shotcrete, the river chamber walls at Emsworth Lock exhibited significant deterioration. Large areas of shotcrete were missing from the landside wall where the smaller tows and pleasure craft using this chamber tie up. Spalling appeared to have originated in the upper portion of the wall where the shotcrete was relatively thin and surface preparation minimal. Spalling apparently propagated down the wall to a point at which the shotcrete was of sufficient thickness (approximately 4 in.) to contain dowels and wire mesh. In comparison, the shotcrete on the riverside wall appeared to be in much better condition.

366. Horizontal cores taken from the chamber walls 14 years after shotcrete resurfacing showed the remaining shotcrete to be in generally good condition. However, the original concrete immediately behind the shotcrete exhibited significant deterioration, probably from cycles of freezing and thawing. Cores of similar concrete from the land chamber which did not receive a shotcrete overlay were in generally good condition from the surface inward. This example is evidence an overlay's contributing to the saturation of the original concrete with increased deterioration from freezing and thawing as a result. Fifty to seventy percent of the existing shotcrete was reported as "drummy" when sounded at approximately 20 years in service. This report would indicate the shotcrete was debonded and remained in place only because of the dowels.

367. Fifteen lock wall monoliths at Dresden Island were resurfaced in 1954 using anchored and reinforced wet-mix shotcrete. In 1976, three horizontal cores were taken through the shotcrete repair sections. These cores showed that the shotcrete had a minimum thickness of 12 in. and exhibited excellent bond to the original concrete. Air-void data determined according to CRD-C 42 indicated the shotcrete had about 3 percent total air with approximately 2 percent of it in voids small enough to be classified as useful for frost resistance. The air-voids spacing factors ranged from 0.010 to 0.014 in. While these values are larger than is desirable (0.008 in. is considered the maximum value for air-entrained concrete), they may have imparted some frost resistance. Also, there were no large voids or strings of voids from lack of consolidation such as have been observed with other shotcrete specimens.

368. Although there is currently some minor spalling at monolith joints and at the interface between the shotcrete and the concrete placed in 1978 when horizontal armor was installed, overall the shotcrete remains in excellent condition after more than 30 years in service. While the air-void system in the shotcrete may have imparted some frost resistance, more likely its durability is the result of the low permeability typical of shotcrete preventing the shotcrete from becoming critically saturated. This low permeability would contribute to increased saturation and potential deterioration of the original concrete if moisture migrated from beneath or behind the shotcrete. However, in this case the thickness of shotcrete apparently exceeded the depth of frost penetration. After more than 40 years in service, the average depth of deterioration in the unrepaired concrete walls as determined by petrographic examination was approximately 8-1/2 in. Assuming that the depth of frost penetration is approximately equal to the depth of deterioration, the 12-in. thickness of shotcrete is about 50 percent greater than the depth of frost penetration. Therefore, the concrete behind the shotcrete would not be expected to exhibit freeze-thaw deterioration even though it may have been critically saturated.

369. Introduction of expansion joint material at lock wall monolith joints during shotcrete resurfacing at Dresden Island is believed to have contributed to spalling at the joints. Since there are no expansion joints in the lock walls, the expansion joint in the resurfaced zone cannot function and only absorbs water which causes spalling through cycles of freezing and

thawing. It was recommended by the District that expansion joints not be used in future resurfacing projects.

Preplaced-Aggregate Concrete

370. Preplaced-aggregate concrete is made by filling forms with coarse aggregate and then filling the voids of the aggregate by pumping in a sand-cement grout. As the grout is pumped into the forms, it fills the voids, displacing any water and forms a concrete mass. Since drying shrinkage and creep occur almost exclusively in the cement paste fraction of concrete and since both phenomena are resisted by the aggregate, particularly if the coarse aggregate particles are in point-to-point contact, drying shrinkage and creep are both remarkably less for preplaced-aggregate concrete than for conventionally placed concrete. This reduction in drying shrinkage reduces the probability of cracking under conditions of restrained shrinkage. The dimensional stability of preplaced-aggregate concrete makes it attractive as a material for the rehabilitation of lock walls and appurtenant structures, particularly if it is successful in mitigating or eliminating the unsightly cracking commonly experienced with conventionally placed concrete. Its potential is further enhanced by the fact that it can be conveniently formed and placed underwater and that it can be grouted in one continuous operation so that there are no cold joints. Early laboratory tests on preplaced-aggregate concrete with a nominal 4 percent entrained air or hydrogen gas indicated its resistance to freezing and thawing, judged by losses in test specimen weight, was superior to air-entrained conventional concrete (Davis 1960). Later work (Tynes and McDonald 1968) indicated that the resistance of conventional preplaced-aggregate concrete to accelerated freezing and thawing at 28 days age, as judged by dynamic modulus of elasticity, was much less than that of concrete containing entrained air both conventionally mixed and preplaced. However, the use of grout containing an air-entraining admixture resulted in preplaced-aggregate concrete with approximately the same resistance to freezing and thawing as conventional air-entrained concrete.

371. The lock chamber walls at Lock No. 5, Monongahela River, were resurfaced in 1950 with preplaced-aggregate concrete. The plans called for removal of approximately 18 in. of old concrete from an area extending from the top of the lock walls to about 18 in. below normal pool elevation and the

refacing of this area with reinforced concrete. Specifications required that the concrete have a minimum 28-day compressive strength of 3,500 psi and provided that concrete could be placed by either conventional or preplaced-aggregate methods. Also, it was required that one of the two lock chambers be open to navigation at all times. The contractor elected to perform the work without constructing cofferdams. This plan necessitated concrete removal and replacement below pool elevation "in the wet." The contractor selected the preplaced-aggregate method for concrete placement.

372. Lock No. 5 was removed from service and, with the exception of the land wall, razed in conjunction with the construction of Maxwell Lock and Dam in 1964. A visual examination of the remaining wall in July 1985 showed that the preplaced-aggregate concrete had some cracking and leaching but overall appeared to be in generally good condition after 35 years exposure. This repair demonstrates that the preplaced-aggregate method of the concrete placement is a practical alternative to conventional methods for refacing lock walls.

373. The low bid ($85,000) for refacing the walls at Lock No. 5, based on the use of preplaced-aggregate concrete, was almost $60,000 lower than the second lowest bid. A 1974 analysis of potential methods for rehabilitation of Marseilles Lock indicated that the cost of preplaced-aggregate concrete was expected to be essentially the same as for conventional cast-in-place concrete. However, recent bid prices indicate that the cost of preplaced-aggregate concrete may be as much as twice that of conventional concrete. The increasing cost is generally attributed to the stronger and tighter form work required and the limited number of contractors with experience in the use of preplaced-aggregate concrete. Therefore, if preplaced-aggregated concrete is the desired repair material, it must be specified uniquely and not as an alternate.

Precast Concrete Stay-in-Place Forms

374. Permanent forming systems of high-quality precast concrete panels appear to have significant potential in minimizing the cracking problems normally encountered in conventional refacing of lock walls. Also, stay-in-place forming systems appear to have potential for minimizing or eliminating the need for closure of a lock chamber during rehabilitation. A permanent form

made of precast concrete slabs was successfully used to reface Barker Dam in the late 1940's. Approximately 1,000 slabs, 8 in. thick, were cast in a variety of sizes to accommodate joint spacings in the old dam. Most of the panels were 6 ft 8 in. wide by about 12 ft long and weighed about 4 tons. In a 1974 analysis of potential methods for rehabilitation of Marseilles Lock, it was estimated that a permanent precast panel forming system would cost more than twice as much as conventional concrete forming and placing. However, the estimated cost of a precast stay-in-place forming system recently designed by ABAM Engineers (1986) is slightly less than conventional concrete replacement. The potential of such a system should be evaluated during the development of plans for any major lock wall rehabilitation.

Thin Overlays

375. Thin overlays of epoxy, epoxy mortar, conventional shotcrete (with and without fiber reinforcement), and latex-modified, fiber-reinforced shotcrete have been used in the case histories described herein. In most cases, they have been used in areas where the depth of deterioration was minimal, and apparently the intent was either to protect the existing concrete or to improve the appearance of the structure. There has been little, if any, concrete removal associated with these applications. Surface preparation techniques used include handchipping, high-pressure water blasting, bush hammering, and sandblasting.

376. The riverside of the river wall, the lower guide wall, all existing gate recesses, and various areas on the upper guide wall and middle wall at Lock No. 3, Monongahela River, were treated with a thin coating of conventional, unreinforced shotcrete. Shotcrete was applied using the dry-mix process with a sand-cement ratio of approximately 4.0. In May 1981, approximately one year following completion of the coating application, several small localized areas of the shotcrete coating were reportedly abraded and spalled, evidently by tows entering and leaving the locks. The shotcrete on the riverside of the river wall, an area not subject to tow impact, was in good condition with only minor shrinkage cracking. It was concluded that the performance of the shotcrete coating in areas subject to tow abrasion and impact was unsatisfactory, particularly where the shotcrete was applied in a very thin layer (less than 1/2 in.) over relatively smooth surfaces, and where

exposure to impact and rubbing by tows was most severe. In June 1982, it was reported that failure of the shotcrete coating appeared to be progressing. Additional failures of the shotcrete coating were reported in July 1985, including spalling within the gate recesses.

377. A conventional, unreinforced shotcrete coating was applied to selected areas of the lock walls at Emsworth Lock. In some areas of surface spalling, this coating was as much as 2-3 in. thick; however, in most cases, it was the minimum 3/4-in. overbuild required over the original concrete surface. After approximately 2-1/2 years in service, the shotcrete coating applied to portions of the river face of six monoliths of the land wall was in generally poor condition. About 80 percent of the shotcrete coating on monoliths L-37 and L-38 between elevation 692 and 703 was missing to depths of up to 2 in. The coating on monolith L-39 contained a reflective vertical crack which was also present in the concrete below the lower limit of the shotcrete coating. The shotcrete coating applied in the gate recesses was in poor condition with numerous horizontal and diagonal cracks. A few cracks were dampened by water, and seepage through several cracks covered large areas of the coating. Small areas of the shotcrete coating were debonded from the underlying concrete.

378. The shotcrete coating applied to the upstream gate recess in the land face of the middle wall was in fair condition. The coating in this recess was in somewhat better condition than the coating in the opposite recess in the land wall. Since the land face of the middle wall is shaded from sunlight, it probably undergoes fewer cycles of freezing and thawing than the opposite wall which is exposed to sunlight. The shotcrete coating in the downstream recess was in fair condition above high pool elevation and in good condition between low and high pool elevations. Again the coating was in better condition than the opposite recess in the landwall.

379. After approximately 3-1/2 years in service, the condition of the shotcrete coating applied to the river face of middle wall monoliths M-1 through M-4 and M-6 through M-9 was described as poor and fair, respectively. Numerous cracks, some with water seepage and soak staining, were reported. The deterioration of the shotcrete coating is thought to be caused by freezing and thawing action, and it will likely continue to a point at which large areas will become debonded. The condition of the shotcrete coating in the lower gate recesses of the river chamber was described as good although minor

fine vertical cracking and minor leaching was reported.

380. The shotcrete coating applied to the river face of the river wall was apparently in good condition after approximately 6 months in service. The vertical section of the wall could not be fully viewed; however the sloped section could be observed and no deficiencies were noted. The shotcrete coating on both faces of the lower guard wall was in good condition. The shotcrete coating on the landside had reddish brown stains as a result of barges rubbing against high spots in the shotcrete, but there were no appreciable signs of abrasion.

381. A section of the upper guide wall of Emsworth Locks was repaired with a 1- to 2-in. overlay of steel fiber reinforced shotcrete (Fibercrete) using the dry-mix process. After only 3 months in service, the Fibercrete exhibited numerous examples of impact failure, abrasion erosion, and delamination. The explanation for the poor performance was that the prepackaged mixture used in the repair contained only 60 lb/cu yd of fibers. It was suggested that a much higher cement content and fiber contents up to 200 lb/cu yd would have improved the performance.

382. Epoxy mortar ranging in thickness from 1/4 to 4 in. was applied to vertical wall surfaces immediately below the top corner protection armor at Marseilles Lock. Delamination of the epoxy mortar was observed within days after the lock was reopened to traffic. Tests indicated the predominant causes of failure were barge impact and poor bond to the existing concrete. Lack of cleaning and smoothness of the underlying material were thought to be the principal factors in producing a poor bond. A thin layer of epoxy grout on top of the wall also exhibited significant cracking and delamination within a short period of time. This deterioration was attributed to thermal incompatibility between the epoxy and the existing concrete.

383. A thin overlay of polymer-modified cementitious mortar was placed on the filling flume deck slab upstream of the powerhouse of Lock No. 3, Monongahela River. After approximately 6 months in service, some areas had extensive cracks while other areas were practically free of cracks. After approximately 18 months in service, the polymer mortar overlay was debonded and buckled in several places.

384. A polymer mortar overlay was applied to the top surfaces of two monoliths in the middle wall at Emsworth Locks. After approximately a year in service, the overlay was reported to be in poor condition with extensive

pattern cracking. These cracks were allowing water ponding on top of the monoliths to infiltrate the underlying concrete as evidenced by water seepage on vertical wall surfaces beneath the corner protection armor.

385. Based on field demonstrations and laboratory evaluations, a 3/8-in. thick, fiberglass-reinforced, latex-modified cement coating was selected for repair of the lock chamber walls at Lower Monumental. The success of the repair was totally dependent upon satisfactory surface preparation; therefore several surface cleaning trials were made to verify that the concrete surface could be satisfactorily cleaned with high-pressure water-jet equipment. Since there was no device or test which could be used to measure the degree of "clean" necessary or actually achieved during surface preparation, the contractor was required to prepare sample areas before proceeding with production cleaning. As a result, agreement was easily reached as to what was an acceptable surface condition, and this standard was used throughout the job.

386. Approximately one year after the fiber-reinforced, latex-modified cement coating was applied, the lock walls were evaluated by soundings, visual examination, and core drilling. Based on soundings of a typical section of one monolith, it was concluded that approximately 99 percent of the coating was fully bonded. Generally, the unbonded areas were about 1 sq ft or less in area and located just below a lift joint or adjacent to a monolith joint. Most, if not all, of the bottom lift of coating, below tailwater, was obviously debonded and some had fallen off the wall. This failure was attributed to either insufficient drying time prior to inundation or reemulsification of the latex from continuous saturation. After two years in service, the trial coatings applied in 1979 were performing poorly.

387. After approximately 3 years in service, several small, isolated debonded areas of the latex-modified, fiber-reinforced shotcrete coating were removed and resprayed in March 1983. Also, a fairly large debonded area on monolith 11 and the failed areas of trail coatings on monolith 9 were resprayed. Equipment and procedures, including surface preparation, were essentially the same as those used in the 1980 repairs with the exception that the latex used in 1983 was styrene butadiene instead of saran. Within 6 months, almost all of the coatings applied in March 1983 had failed. Generally, failures occurred within the concrete substrate immediately behind the concrete-shotcrete interface. The remainder of the lock wall coatings were reported to

be in very good condition with the exception of the trial coatings in which an estimated 40 percent of the trial area had failed.

388. Currently, it is reported that an estimated 15 to 20 percent of the latex-modified, fiber-reinforced shotcrete applied in 1980 was debonded following the winter of 1985-86. Also, a large piece of the coating had fallen off, striking a barge inside the lock chamber.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

389. Approximately half of the Corps 269 navigation lock chambers were built prior to 1940; in fact, the average age of these older structures is nearly 67 years or well beyond their 50-year design life. Many of these structures exhibit significant deterioration and are in need of rehabilitation or replacement. Since rehabilitation costs are usually one-tenth to one-fourth that of replacement, rehabilitation is very attractive from a financial standpoint. In the limited number of lock rehabilitations to date, the typical approach has been to use principles normally associated with new construction. However, there is increasing evidence that rehabilitation work is often more complex and that normal new construction procedures often do not produce satisfactory results in rehabilitation work. Obviously, in a rehabilitation program which could ultimately cost more than $2 billion, there is a need for development of new and innovative technology to ensure optimum utilization of available resources.

390. The general approach in lock wall rehabilitation has been to remove the deteriorated nonair-entrained concrete and replace it with new air-entrained concrete or shotcrete. Explosive blasting has been successfully used at several Corps projects and appears to be the most cost-effective and expedient means for removing large quantities of concrete. Removal by blasting has been generally prohibited in sensitive areas such as gate monoliths for fear of damage to the remaining concrete around gate anchorages. The need for such restrictions should be investigated, and if, in fact, they are necessary, acceptable alternatives to blasting should be developed.

391. Once the deteriorated concrete has been removed, conventional cast-in-place concrete has been used as the replacement material in most lock wall rehabilitation projects. Conventional concrete replacement has several advantages over other rehabilitation techniques including: (a) replacement concrete mixtures can be proportioned to simulate the existing concrete substrate, thus minimizing strains caused by material incompatibility; (b) proper air entrainment in the replacement concrete can be obtained by use of admixtures to ensure resistance to cycles of freezing and thawing; and (c) materials, equipment, and personnel with experience in conventional concrete application are readily available in most areas.

392. One of the most persistent problems in lock wall rehabilitation

with this approach is cracking in the replacement concrete. These cracks, which generally extend completely through the conventional replacement concrete, are attributed primarily to restraint of volume changes resulting from shrinkage, thermal gradients, and autogenous volume changes. In most cases, such cracking will not cause structural deficiencies. However, the cracks are unsightly and may require additional maintenance to minimize deterioration.

393. Any variations in concrete materials, mixture proportions, and construction procedures that will minimize shrinkage or reduce concrete temperature differentials should be considered. If weather conditions on the day of placement are conducive to plastic shrinkage cracking (ACI 1985), appropriate actions such as erecting windbreaks, erecting shade over the placement, cooling the concrete, and misting should be taken after placement. Additionally, it will be beneficial to minimize the loss of moisture from the concrete surface between placing and finishing. Finally, curing should be started as soon as practical. The general approach to prevention of drying shrinkage is either to reduce the tendency of the concrete to shrink or to reduce the restraint, or both. The following will help to reduce the tendency to shrink: using less water in the concrete; using larger aggregate to minimize paste content; placing the concrete at as low a temperature as is practical; dampening the subgrade and the forms; dampening aggregates if they are dry and absorptive; using proper curing procedures; and providing an adequate amount of reinforcement to distribute and reduce the size of cracks that do occur. To reduce restraint, provide adequate contraction joints should be provided.

394. In general, the following may be beneficial in controlling internally generated temperature differences: using as low a cement content as possible; using a low heat of hydration cement or a combination of cement and pozzolans; placing the concrete at the minimum practical temperature; selecting aggregates with low moduli of elasticity and low coefficients of thermal expansion; cooling or insulating the placement as appropriate to minimize temperature differentials; and minimizing the effects of stress concentrators that may instigate cracking. Concrete cracking caused by externally generated temperature differences is best controlled by the use of contraction and expansion joints. Providing reinforcing steel (temperature steel) will help to distribute cracks and minimize the size of those that do occur.

395. Since drying shrinkage occurs almost exclusively in the cement paste fraction of concrete and is resisted by the aggregate, particularly if

the coarse aggregate particles are in point-to-point contact, drying shrinkage for preplaced-aggregate concrete is significantly less than for conventionally placed concrete. This reduction in drying shrinkage reduces the probability of cracking under conditions of restrained shrinkage and makes preplaced-aggregate concrete an attractive alternative to conventional concrete in lock wall rehabilitation. Its potential is further enhanced by the fact that it can be conveniently formed and placed underwater.

396. Precast concrete panels used as stay-in-place forms also appear to have significant potential in minimizing the cracking problems normally encountered in conventional refacing of lock walls. Precast concrete can be produced under tightly controlled conditions, so such panels should provide a wall surface of superior durability with minimal cracking. Another advantage of this system is the potential for minimizing or eliminating the need for closure of the lock during rehabilitation. Current work to demonstrate the constructibility of the stay-in-place forming system should be expedited.

397. With the exception of Dresden Island Lock where the minimum thickness of shotcrete was about 12 in., shotcrete has generally performed rather poorly when used in lock wall repairs. This failure is particularly true when thin layers of conventional shotcrete are subjected to impact and abrasion during normal lock operations. While the addition of fibers and latex improved performance, significant failures still occurred within 5 years after application. The poor performance of shotcrete is primarily attributed to its typical low permeability which increases the potential for critical saturation of the original concrete from moisture migration from behind or beneath the shotcrete. Subsequent cycles of freezing and thawing under such conditions cause deterioration of the original concrete immediately behind the relatively thin overlay of shotcrete. The result is debonding of the shotcrete overlay. The same phenomena has occurred where relatively thin (6-in. thickness or less) overlays of concrete were used. It appears that if the thickness of any repair section is less than the depth of frost penetration, freeze-thaw deterioration of the existing nonair-entrained concrete should be expected. Appropriate laboratory test methods and apparatus should be developed for evaluation of repair materials under simulated lock resurfacing conditions.

398. In design of future lock wall rehabilitations, the cost of alternative repairs should be carefully evaluated in relation to the desired service life of the rehabilitated structure. Only a few years of good service

at best should be expected of shotcrete in relatively thin layers. However, the cost of such repair will be relatively low. In comparison, conventionally formed and placed concrete, shotcrete, and preplaced-aggregate concrete, each properly proportioned and placed in thicknesses greater than the depth of frost penetration, should provide a minimum of 25 years of service but at successively greater initial costs. Precast concrete panels used as stay-in-place forms should provide even greater durability at approximately the same cost as conventional replacement concrete.

399. Some general conclusions and recommendations based on this review of previous lock wall repairs have been formulated. However, before detailed guidance can be developed, a number of questions must be addressed.

a. Can deterioration caused by cycles of freezing and thawing of nonair-entrained concrete below an overlay be controlled?

b. What are the consequences of such deterioration?

c. Is it feasible to provide drainage within underlying concrete or to impregnate it with a protective material to minimize moisture intrusion?

d. Should the depth of a frost penetration with a given lock wall indicate the thickness of the repair overlay?

e. Would provision of a frost barrier at the interface between the existing concrete and the overlay be beneficial?

f. What design and construction controls are required to ensure that the repair material is compatible with the existing concrete?

g. Is it feasible to eliminate the restraint provided through bond of the repair material to the existing concrete?

h. Is it feasible to use precast concrete stay-in-place forms to eliminate cracking problems and minimize closure times associated with lock rehabilitation?

Most of these questions are being investigated as part of the REMR Research Program, and results of the investigation will be reported as they become available.

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APPENDIX A

ABSTRACTS OF BIDS FOR SELECTED REHABILITATION PROJECTS

Table A1

Abstract of Bids for Emsworth Locks and Dams Rehabilitation

VITATION FOR BIDS NUMBER: DACW59-81-B-0048

DATE ISSUED: 5 June 1981

DATE OPENED: 18 September 1981

ISSUING OFFICE: U.S. Army Engr. Dist., Pgh. Corps of Engineers Federal Building 1000 Liberty Avenue Pittsburgh, PA 15222

FOR: Locks and Dams Rehabilitation Emsworth Locks and Dams Ohio River, Pennsylvania

NUMBER OF ADDENDA ISSUED: Eleven

GOVERNMENT ESTIMATE: [ ] HIRED LABOR ESTIMATE; [X] REASONABLE CONTRACT ESTIMATE (without profit); [ ] REASONABLE CONTRACT ESTIMATE (including profit)

CERTIFICATE

I CERTIFY that I have opened, read, and recorded herein all bids received in response to this invitation.

JOHN L. RICHARDS, LTC — SIGNATURE John L. Richards — Corps of Engineers.

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	BID NO. 2 HASSMAN CONSTRUCTION CO 8901 State Line P.O. Box 8458 Kansas City, MO 64114		BID NO. 3 MORRISON-KNUDSEN Co., INC. 777 Post Road Darien, CT 06820		BID NO. 6 FRUIN-COLNON CONTRACTING COMPANY 9666 Olive Boulevard Saint Louis, MO 63132	
						BID SECURITY: Bid Bond		BID SECURITY: Bid Bond		BID SECURITY: Bid Bond	
						ADDENDA ACKNOWLEDGED: Yes		ADDENDA ACKNOWLEDGED: Yes		ADDENDA ACKNOWLEDGED: Yes	
				UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
1.	Mobilization and Preparatory Work	1	Job	Sum	745,000.00	Sum	1,364,000.00	Sum	667,539.00	Sum	2,000,000.00
2. a.	Switch Boats No. 1	60	days	9,400.00	564,000.00	2,600.00	156,000.00	3,600.00	216,000.00	3,600.00	216,000.00
b.	No. 2	60	days	9,400.00	564,000.00	2,600.00	156,000.00	3,600.00	216,000.00	3,600.00	216,000.00
3.	Government Field Office	1	Job	Sum	145,000.00	Sum	175,000.00	Sum	80,000.00	Sum	100,000.00
4. a.	Dewatering Locks and Pumping River Chamber	1	Job	Sum	261,000.00	Sum	1,100,000.00	Sum	500,000.00	Sum	200,000.00
b.	Land Chamber	2	Ea.	73,000.00	146,000.00	100,000.00	200,000.00	100,000.00	200,000.00	40,000.00	80,000.00
5.	Removal of Equipment, Miscellaneous Metal and Guard Fence	1	Job	Sum	38,000.00	Sum	200,000.00	Sum	100,000.00	Sum	50,000.00
6. a.	Removal of Concrete Faces of Lock Walls	2,230	Sq. Yd	220.00	490,600.00	180.00	401,400.00	25.00	55,750.00	60.00	133,800.00
b.	Top of Lock Walls	520	Sq. Yd	65.00	33,800.00	440.00	228,800.00	36.00	18,720.00	300.00	156,000.00
c.	Cell Cap	250	Cu. Yd	140.00	35,000.00	100.00	25,000.00	30.00	7,500.00	80.00	20,000.00
d.	Esplanade	525	Cu. Yd	75.00	39,375.00	200.00	105,000.00	26.00	13,650.00	40.00	21,000.00

DD FORM 1 OCT 75 1501-1 EDITION OF 1 NOV 64 IS OBSOLETE

(Continued)

(Sheet 1 of 9)

Table A1 (Continued)

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	Bid No. 2 UNIT PRICE	Bid No. 2 ESTIMATED AMOUNT	Bid No. 3 UNIT PRICE	Bid No. 3 ESTIMATED AMOUNT	Bid No. 6 UNIT PRICE	Bid No. 6 ESTIMATED AMOUNT
e.	Docking Area	7	Cu. Yd	350.00	2,450.00	1,000.00	7,000.00	330.00	2,310.00	500.00	3,500.00
f.	Recess and Other Miscellaneous Concrete	540	Cu. Yd	370.00	199,800.00	500.00	270,000.00	180.00	97,200.00	1,200.00	648,000.00
7.	Drilling for Floating Mooring Bitt										
a.	48" Ø	40	Lin. Ft	860.00	34,400.00	800.00	32,000.00	1,200.00	48,000.00	1,000.00	40,000.00
b.	30" Ø	81	Lin. Ft	610.00	49,410.00	500.00	40,500.00	700.00	56,700.00	600.00	48,600.00
8. a.	Removal of Structures Land Wall Power House	1	Job	Sum	12,200.00	Sum	10,000.00	Sum	35,100.00	Sum	40,000.00
b.	Middle Wall Operations Building	1	Job	Sum	9,100.00	Sum	10,000.00	Sum	15,000.00	Sum	15,000.00
c.	Control Station Shelters	1	Job	Sum	4,800.00	Sum	1,500.00	Sum	5,000.00	Sum	15,000.00
d.	Maintenance Shed	1	Job	Sum	4,700.00	Sum	1,000.00	Sum	10,000.00	Sum	3,000.00
e.	Mooring Piers	1	Job	Sum	10,500.00	Sum	30,000.00	Sum	50,000.00	Sum	50,000.00
f.	Upper Guard Wall Cell No. 1 (Piling only)	1	Job	Sum	18,400.00	Sum	45,000.00	Sum	75,000.00	Sum	50,000.00
g.	Railroad Siding	1	Job	Sum	6,200.00	Sum	20,000.00	Sum	10,000.00	Sum	8,000.00
9.	Removal of Cell Mill	1,750	Cu. Yd.	5.00	8,750.00	10.00	17,500.00	3.00	5,250.00	3.50	6,125.00
10.	Drilling Holes for Rock Anchors and Reinforcing Bars										
a.	5-1/2" dia. (Type A Anchors)	5,535	Lin. Ft	49.00	271,215.00	40.00	221,400.00	60.00	332,100.00	45.00	249,075.00
b.	7-3/4" dia. (Type B Anchors)	2,610	Lin. Ft	80.00	208,800.00	90.00	234,900.00	120.00	313,200.00	95.00	247,950.00
c.	Pilot Holes and Casing on River Wall	120	Lin. Ft	100.00	12,000.00	150.00	18,000.00	200.00	24,000.00	151.00	18,120.00
d.	Holes and Casing Throug Overburden at Esplanade	1,030	Lin. Ft	35.00	36,050.00	25.00	25,750.00	30.00	30,900.00	25.00	25,750.00
e.	2-1/2" dia. (#9 bars)	370	Lin. Ft	18.00	6,660.00	15.00	5,550.00	20.00	7,400.00	15.00	5,550.00
f.	3" dia. (#11 bars)	950	Lin. Ft	43.00	40,850.00	35.00	33,250.00	45.00	42,750.00	35.00	33,250.00
g.	Holes for Resin Grouted Anchors	675	Lin. Ft	26.00	17,550.00	22.00	14,850.00	25.00	16,875.00	23.00	15,525.00
11.	Cement for Pressure Grouting	105	Bags	24.00	2,520.00	6.00	630.00	7.00	735.00	30.00	3,150.00

(Continued)

(Sheet 2 of 9)

Table A1 (Continued)

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	Bid No. 2 UNIT PRICE	Bid No. 2 ESTIMATED AMOUNT	Bid No. 3 UNIT PRICE	Bid No. 3 ESTIMATED AMOUNT	Bid No. 6 UNIT PRICE	Bid No. 6 ESTIMATED AMOUNT
12. a.	Redrilling Through Grout 5-1/2" dia.	490	Lin.Ft.	30.00	14,700.00	25.00	12,250.00	30.00	14,700.00	25.00	12,250.00
b.	7-3/4" dia.	150	Lin.Ft.	43.00	6,450.00	45.00	6,750.00	50.00	7,500.00	45.00	6,750.00
13.	Rock Anchors and Reinforcing Bars										
a.	Type "A" Anchors	4,640	Lin.Ft.	46.00	213,440.00	50.00	232,000.00	25.00	116,000.00	13.00	60,320.00
b.	Type "B" Anchors	1,760	Lin.Ft.	89.00	156,640.00	65.00	114,400.00	50.00	88,000.00	28.00	49,280.00
c.	Test Anchors	310	Lin.Ft.	75.00	23,250.00	85.00	26,350.00	81.50	25,265.00	74.00	22,940.00
d.	#9 Reinforcing Bars	370	Lin.Ft.	39.00	14,430.00	100.00	37,000.00	21.00	7,770.00	25.00	9,250.00
e.	#11 Reinforcing Bars	950	Lin.Ft.	42.00	39,900.00	45.00	42,750.00	22.00	20,900.00	28.00	26,600.00
f.	Sill Anchors	675	Lin.Ft.	60.00	40,500.00	50.00	33,750.00	23.00	15,525.00	170.00	114,750.00
14.	Steel Sheet Piling, Government Furnished, for Cells	32,800	Sq.Ft.	7.00	229,600.00	18.00	590,400.00	7.00	229,600.00	11.00	360,800.00
15.	Splices	20	Ea.	75.00	1,500.00	75.00	1,500.00	75.00	1,500.00	75.00	1,500.00
16.	Sand and Gravel Fill	1,215	Cu.Yd.	5.00	6,075.00	6.00	7,290.00	3.00	3,645.00	3.00	3,645.00
17.	Storm Drainage System	1	Job	Sum	107,000.00	Sum	300,000.00	Sum	45,000.00	Sum	70,000.00
18.	Access Road	1	Job	Sum	192,000.00	Sum	300,000.00	Sum	125,000.00	Sum	90,000.00
19. a.	Chain link Fencing 6' high	250	Lin.Ft.	40.00	10,000.00	27.00	6,750.00	21.50	5,375.00	11.00	2,750.00
b.	3' high	110	Lin.Ft.	20.00	2,200.00	24.00	2,640.00	14.00	1,540.00	8.00	880.00
20. a.	Concrete Reface Lock Walls in Chamber	820	Sq.Yd.	310.00	254,200.00	400.00	328,000.00	300.00	246,000.00	310.00	254,200.00
b.	Reface Lock Walls Outside Chamber	1,260	Sq.Yd.	305.00	384,300.00	400.00	504,000.00	325.00	409,500.00	260.00	327,600.00
c.	Resurface Top of Walls, 8-inch Depth	270	Sq.Yd.	78.00	21,060.00	200.00	54,000.00	73.00	19,710.00	260.00	70,200.00
d.	Resurface Top of Walls, 12-inch Depth	2,900	Sq.Yd.	75.00	217,500.00	200.00	580,000.00	135.00	391,500.00	260.00	754,000.00
e.	Gate Monolith	325	Cu.Yd.	315.00	102,375.00	400.00	130,000.00	300.00	97,500.00	510.00	165,750.00
f.	Tremie (for cells)	1,790	Cu.Yd.	152.00	272,080.00	150.00	268,500.00	100.00	179,000.00	100.00	179,000.00

(Continued)

(Sheet 3 of 9)

Table A1 (Continued).

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	Bid No. 2 UNIT PRICE	Bid No. 2 ESTIMATED AMOUNT	Bid No. 3 UNIT PRICE	Bid No. 3 ESTIMATED AMOUNT	Bid No. 6 UNIT PRICE	Bid No. 6 ESTIMATED AMOUNT
g.	Esplanade Paving	445	Cu.Yd.	207.00	92,115.00	400.00	178,000.00	400.00	178,000.00	210.00	93,450.00
h.	Recess and Other Miscellaneous Concrete	430	Cu.Yd.	340.00	146,200.00	1,000.00	430,000.00	600.00	258,000.00	810.00	348,300.00
i.	Fill	1,860	Cu.Yd.	140.00	260,400.00	150.00	279,000.00	120.00	223,200.00	80.00	148,800.00
j.	Cap	230	Cu.Yd	285.00	65,550.00	150.00	34,500.00	200.00	46,000.00	210.00	48,300.00
21.	Not Used										
22. a.	Steel Reinforcement Reinforcement Steel	193,800	Lbs.	.74	143,412.00	1.00	193,800.00	1.00	193,800.00	1.30	251,940.00
b.	Welded Wire Fabric	22,000	Lbs.	1.35	29,700.00	1.00	22,000.00	1.15	25,300.00	1.60	35,200.00
c.	Dowels	58,800	Lbs.	.90	52,920.00	.25	14,700.00	.60	35,280.00	.50	29,400.00
23.	Compacted Base Course	840	Cu.Yd.	38.00	31,920.00	9.00	7,560.00	24.00	20,160.00	16.00	13,440.00
24.	2" dia. Trench Drains	175	Lin.Ft.	24.00	4,200.00	20.00	3,500.00	21.00	3,675.00	30.00	5,250.00
25.	Misscellaneous Concrete Work										
a.	Alterations to Land Wall Culvert	1	Job	Sum	109,000.00	Sum	100,000.00	Sum	50,000.00	Sum	200,000.00
b.	New Filling Ports	9	Ea.	10,000.00	90,000.00	30,000.00	270,000.00	10,000.00	90,000.00	17,000.00	153,000.00
c.	Sealing Crossover Construction Joints	1	Job	Sum	20,000.00	Sum	25,000.00	Sum	12,600.00	Sum	11,000.00
d.	Polymer Repair Mortar	180	Cu.Ft.	175.00	31,500.00	325.00	58,500.00	200.00	36,000.00	155.00	27,900.00
26.	Drill Holes and Grout Dowels and Anchors	24,750	Lin.Ft.	12.00	297,000.00	30.00	742,500.00	20.00	495,000.00	14.00	346,500.00
27. a.	Shotcrete Coating	55,810	Sq.Ft.	16.00	892,960.00	9.00	502,290.00	7.00	390,670.00	18.20	1,015,742.00
b.	Repair	3,665	Sq.Ft.	21.00	76,965.00	50.00	183,250.00	20.00	73,300.00	40.00	146,600.00
c.	Monolith Joint Repair	1,660	Lin.Ft.	30.00	49,800.00	110.00	182,600.00	100.00	166,000.00	100.00	166,000.00
28.	Furnish and Install Miscellaneous Metal	185,300	Lbs.	3.50	648,550.00	3.40	630,020.00	3.50	648,550.00	2.50	463,250.00

(Continued)

(Sheet 4 of 9)

Table A1 (Continued)

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	Bid No. 2 UNIT PRICE	Bid No. 2 ESTIMATED AMOUNT	Bid No. 3 UNIT PRICE	Bid No. 3 ESTIMATED AMOUNT	Bid No. 6 UNIT PRICE	Bid No. 6 ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
29.	Furnish and Install Aluminum Planking and Cover Plates										
a.	Aluminum Planking	10,365	Sq.Ft.	16.00	165,840.00	27.00	279,855.00	22.00	228,030.00	22.00	228,030.00
b.	Cover Plates	205	Sq.Ft.	23.00	4,715.00	65.00	13,325.00	67.00	13,735.00	40.00	8,200.00
30.	Furnish and Install Aluminum Rabbet Angles	4,080	Lin.Ft.	7.85	32,028.00	22.00	89,760.00	14.00	57,120.00	13.00	53,040.00
31.	Furnish and Install Guard Fence and Handrail										
a.	Guard Fence	7,690	Lin.Ft.	17.00	130,730.00	45.00	346,050.00	40.00	307,600.00	21.00	161,490.00
b.	1½" dia. Handrail	60	Lin.Ft.	33.00	1,980.00	67.00	4,020.00	82.00	4,920.00	95.00	5,700.00
c.	2" dia. Handrail	480	Lin.Ft.	32.00	15,360.00	67.00	32,160.00	67.00	32,160.00	91.00	43,680.00
32.	Furnish and Install Floating Mooring Bitt	1	Job	Sum	40,000.00	Sum	50,000.00	Sum	84,400.00	Sum	37,500.00
33.	Wall Armor										
a.	Straight Run (Government-Furnished)	171,000	Lbs.	1.00	171,000.00	2.60	444,600.00	1.25	213,750.00	1.20	205,200.00
b.	Straight Run (Contractor-Furnished)	46,400	Lbs.	1.70	78,880.00	3.40	157,760.00	1.75	81,200.00	1.50	69,600.00
c.	Corner Protection	227,700	Lbs.	1.65	375,705.00	2.70	614,790.00	2.00	455,400.00	1.50	341,550.00
34.	Furnish and Install Distance Markers	45	Ea.	70.00	3,150.00	175.00	7,875.00	140.00	6,300.00	90.00	4,050.00
35. a.	Miter Sill Repairs River Chamber	1	Job	Sum	20,000.00	Sum	45,000.00	Sum	20,000.00	Sum	100,000.00
36. a.	Quoin Seal Replacement, River Chamber	1	Job	Sum	40,000.00	Sum	50,000.00	Sum	100,000.00	Sum	200,000.00
b.	Modification, Land Chamber	1	Job	Sum	55,000.00	Sum	65,000.00	Sum	150,000.00	Sum	50,000.00
37.	Pintle Replacement, River Chamber	1	Job	Sum	20,000.00	Sum	32,000.00	Sum	55,000.00	Sum	100,000.00
38.	Rehabilitation of Miter Gate	1	Job	Sum	348,000.00	Sum	300,000.00	Sum	500,000.00	Sum	360,000.00
39.	Painting Existing Misc. Metal	1	Job	Sum	35,700.00	Sum	1,500.00	Sum	7,500.00	Sum	20,000.00
40.	Service Building	1	Job	Sum	385,000.00	Sum	275,000.00	Sum	500,000.00	Sum	600,000.00
41.	Operation Building (Middle Wall)	1	Job	Sum	85,000.00	Sum	55,000.00	Sum	115,000.00	Sum	50,000.00

(Continued)

(Sheet 5 of 9)

Table A1 (Continued)

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	Bid No. 2 UNIT PRICE	Bid No. 2 ESTIMATED AMOUNT	Bid No. 3 UNIT PRICE	Bid No. 3 ESTIMATED AMOUNT	Bid No. 6 UNIT PRICE	Bid No. 6 ESTIMATED AMOUNT
42. a.	Control Station Shelters No. 1	1	Job	Sum	18,000.00	Sum	5,000.00	Sum	20,000.00	Sum	20,000.00
b.	Nos. 2 and 3	2	Ea.	10,000.00	28,000.00	5,000.00	10,000.00	15,000.00	30,000.00	20,000.00	40,000.00
43.	Operation Building (River Wall)	1	Job	Sum	63,000 00	Sum	50,000.00	Sum	75,000.00	Sum	30,000.00
44.	Furnish and Install Hydraulic Piping System, Oil Pumps, Hydraulic Oil Tank and Hydraulic Control Equipment	1	Job	Sum	768,000.00	Sum	600,000.00	Sum	1,215,000.00	Sum	500,000.00
45.	Furnish and Install Compressed Air Piping System	1	Job	Sum	163,000.00	Sum	410,000.00	Sum	333,000.00	Sum	415,000.00
46.	Furnish and Install Service Water Piping System and Pump	1	Job	Sum	66,000.00	Sum	165,000.00	Sum	145,000.00	Sum	173,000.00
47.	Furnish and Install Domestic Water System	1	Job	Sum	11,500.00	Sum	40,000.00	Sum	31,000.00	Sum	42,000.00
48.	Furnish and Install Common Water & Air Piping Systems	1	Job	Sum	23,000.00	Sum	75,000.00	Sum	55,000.00	Sum	72,000.00
49.	Furnish and Install Sanitary Sewer Piping System and Pump Stations	1	Job	Sum	38,000.00	Sum	85,000.00	Sum	78,000.00	Sum	86,000.00
50.	Furnish and Install Gas Piping System	1	Job	Sum	6,500.00	Sum	25,000.00	Sum	24,000.00	Sum	26,000.00
51.	Furnish and Install Gate Operating Machinery										
a.	56' Chamber	4	Ea.	62,000.00	248,000.00	85,000.00	340,000.00	100,000.00	400,000.00	85,000.00	340,000.00
b.	110' Chamber	4	Ea.	150,000.00	600,000.00	140,000.00	560,000.00	175,000.00	700,000.00	160,000.00	640,000.00
52.	Furnish and Install Butterfly Valve Assemblies Total	37	Ea.	23,000.00	851,000.00	15,000.00	555,000.00	20,000.00	740,000.00	33,500.00	1,239,500.00
53.	Furnish Butterfly Valve Assemblies, Insall Machinery cylinders and cylinder [illegible] only	6	Ea.	17,000.00	102,000.00	15,000.00	90,000.00	21,000.00	126,000.00	22,500.00	135,000.00
54.	Furnish and Install Slide Gate and Operating Machinery	1	Job	Sum	168,000.00	Sum	140,000.00	Sum	235,000.00	Sum	157,000.00
55.	Air Compressor Unit	1	Job	Sum	27,000.00	Sum	40,000.00	Sum	42,000.00	Sum	38,000.00
56.	Furnish and Install Tow Manlage and Retriever System	1	Job	Sum	275,000.00	Sum	400,000.00	Sum	300,000.00	Sum	170,000.00
57.	Remove and Replace Timber Fender System	1	Job	Sum	18,000.00	Sum	40,000.00	Sum	17,500.00	Sum	12,000.00
58.	Furnish Maintenance Bulkhead	1	Job	Sum	32,000.00	Sum	20,000.00	Sum	25,000.00	Sum	10,000.00

(Continued)

(Sheet 6 of 9)

Table A1 (Continued)

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	Bid No. 2 UNIT PRICE	Bid No. 2 ESTIMATED AMOUNT	Bid No. 3 UNIT PRICE	Bid No. 3 ESTIMATED AMOUNT	Bid No. 6 UNIT PRICE	Bid No. 6 ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
59.	Furnish and Install Pipe Supports	2,500	Lbs.	7.00	17,500.00	20.00	50,000.00	30.00	75,000.00	20.00	50,000.00
60.	Replacement of Roofs on Bulkhead Storage Pile	1	Job	Sum	17,400.00	Sum	30,000.00	Sum	25,000.00	Sum	20,000.00
61.	Excavation (Underwater)	35,460	Cu. Yd.	7.50	265,950.00	8.00	283,680.00	6.00	212,760.00	14.00	496,440.00
62.	Drilling Holes For Tremie Concrete	645	Lin. Ft.	90.00	58,050.00	40.00	25,800.00	50.00	32,250.00	40.00	25,800.00
63.	Drilling NX Cores	185	Lin. Ft.	50.00	9,250.00	25.00	4,625.00	35.00	6,475.00	25.00	4,625.00
64.	Tremie Concrete For Dam Aprons and Abutment										
a.	First 1000 Cu. Yds.	1,000	Cu. Yd.	140.00	140,000.00	90.00	90,000.00	220.00	220,000.00	100.00	100,000.00
b.	Over 1000 Cu. Yds.	700	Cu. Yd.	115.00	80,500.00	90.00	63,000.00	130.00	91,000.00	75.00	52,500.00
65.	Filter Cloth	17,000	Sq. Yds.	10.00	170,000.00	9.00	153,000.00	12.00	204,000.00	9.00	153,000.00
66. a.	Erosion Protection Bedding Material	7,600	Tons	24.00	182,400.00	16.00	121,600.00	18.00	136,800.00	17.00	129,200.00
b.	OPTION NO. 1 (1) Graded Stone	38,200	Tons	26.00	993,200.00	14.00	534,800.00	18.00	687,600.00	15.00	573,000.00
	(2) Stone Overlay	41,400	Tons	30.00	1,242,000.00	18.00	745,200.00	25.00	1,035,000.00	21.00	869,400.00
66. 67.	OPTION No. 2-No Bid Abutment Piling										
a.	Steel Sheet Piling (Government Furnished)	1,570	Sq. Ft.	22.00	34,540.00	25.00	39,250.00	12.50	19,625.00	16.00	25,120.00
b.	Master Piles	330	Lin. Ft.	100.00	33,000.00	85.00	28,050.00	50.00	16,500.00	65.00	21,450.00
c.	Fabricated Piles	66	Lin. Ft.	125.00	8,250.00	140.00	9,240.00	70.00	4,620.00	40.00	2,640.00
68.	Alterations to Dam Gate Sills	8	Ea.	35,000.00	280,000.00	45,000.00	360,000.00	40,000.00	320,000.00	50,000.00	400,000.00
69.	Scaling Service Bridges	2,200	Sq. Yd.	20.00	44,000.00	9.00	19,800.00	10.00	22,000.00	21.00	46,200.00
70.	Cleaning and Protection of Bridge Seats	1	Job	Sum	40,000.00	Sum	10,000.00	Sum	15,000.00	Sum	37,000.00
71. a.	Renovation of Dam Gates Vertical Lift Gates										
	(1) Replace Side Seal Assemblies	1	Job	Sum	240,000.00	Sum	265,000.00	Sum	300,000.00	Sum	110,000.00

(Continued)

(Sheet 7 of 9)

Table A1 (Continued)

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
				Bid No. 2				Bid No. 3		Bid No. 6	
	(2) Miscellaneous Metal Work (a) First 138,000 Lbs.	138,000	Lbs.	8.00	1,104,000.00	5.00	690,000.00	10.00	1,380,000.00	8.00	1,104,000.00
	(b) Over 138,000 Lbs.	14,000	Lbs.	6.50	91,000.00	5.00	70,000.00	4.00	56,000.00	6.50	91,000.00
	(3) Rivet Replacement	18,000	Ea.	17.00	309,400.00	19.00	345,800.00	15.00	273,000.00	24.00	436,800.00
b.	Sidney-Type Gate										
	(1) Replace Bottom and Side Seals	1	Job	Sum	53,000.00	Sum	140,000.00	Sum	50,000.00	Sum	37,000.00
	(2) Miscellaneous Metal Work (a) First 3,000 Lbs.	3,000	Lbs.	5.00	15,000.00	5.00	15,000.00	10.00	30,000.00	18.00	54,000.00
	(b) Over 3,000 Lbs.	1,000	Lbs.	4.50	4,500.00	5.00	5,000.00	4.00	4,000.00	10.00	10,000.00
	(4) Rivet Replacement	2,000	Ea.	13.00	26,000.00	19.00	38,000.00	15.00	30,000.00	17.00	34,000.00
72.	Modification of Bulkheads	1	Job	Sum	42,000.00	Sum	110,000.00	Sum	150,000.00	Sum	130,000.00
73.	Painting										
a.	Painting Service, Bridges and Bulkhead, Storage Pits	1	Job	Sum	250,000.00	Sum	300,000.00	Sum	400,000.00	Sum	170,000.00
b.	Painting Dam Gates										
	(1) Gates Nos. 1, 2, 3, 4, 5 6, 10 and 11	8	Ea.	40,000.00	320,000.00	50,000.00	400,000.00	70,000.00	560,000.00	55,000.00	440,000.00
	(2) Gate No. 9 (Sidney-Type)	1	Job	Sum	26,000.00	Sum	30,000.00	Sum	40,000.00	Sum	33,000.00
	(3) Gates Nos. 7, 8, 12, 13, and 14	8,000	Sq Ft	6.00	48,000.00	5.00	40,000.00	5.00	40,000.00	6.25	50,000.00
c.	Regalvanize Service Bridge Grating	1	Job	Sum	14,000.00	Sum	26,000.00	Sum	7,000.00	Sum	60,000.00
74.	Electrical Work	1	Job	Sum	815,000.00	Sum	1,000,000.00	Sum	1,000,000.00	Sum	564,653.00
75.	Diesel Electric Generator Unit	1	Job	Sum	50,000.00	Sum	65,000.00	Sum	70,000.00	Sum	66,000.00
76.	Hydroelectric Power Plant	1	Job	Sum	635,000.00	Sum	700,000.00	Sum	1,000,000.00	Sum	800,000.00
	TOTAL Items 1 thru 76 Inclusive				$22,542,550.00		25,907,120.00		24,285,989.00		$24,785,300.00

(Continued)

(Sheet 8 of 9)

Table A1 (Concluded)

Abstract of Bids - Construction

ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
	ADDITIONAL BIDS:					INFORMALITIES (cont.)					
	Bid No. 1 - Al Johnson Construction Co., 1700 Northwestern Financial Center, Minneapolis, Minnesota 55402 $26,205,460.00					Bid No.	2 - Amount of total price bid changed from "$25,907,595.00 to $25,907,120.00." Plant & Equipment Schedule not completed.				
	Bid No. 4 - John F. Casey Company, P.O. Box 1[illegible]3, Pittsburgh, PA 15230 $31,845,725.00					Bid No.	3 - Certification re Percent Foreign Content & Clean Air & Water Certification not complete on SF-19-B. Bidder states address as 777 Post Rd., Darien, CT 06820, on bid & SF-19-B; states address as One Harrison-Knudsen Plaza, Boise Idaho 83729 on Bid Bond. Date of bid not shown on Bid Bond.				
	Bid No. 5 - Mergentime Corporation, Box 662, Barbles Corner Road Flemington, New Jersey 08822 $27,624,607.00										
	INFORMALITIES:					Bid No.	4 - DUNS No. not inserted.				
	Bid No. 1 - Amount of Item 79.[illegible] corrected from "$177,000.00 to "$179,000.00" thereby changing the total price bid from "$26,203,460.00 to $26,205,460.00".					Bid No.	5 - County not included in address on reverse of bid.				
	Certification re Percent Foreign Content on SF 19-B not completed.										
	Plant & Equipment Schedule not completed-Bidder states: "Adequate Equipment to Construct the job."										

(Sheet 9 of 9)

Table A2

Abstract of Bids for Stage I Rehabilitation, Lockport Lock

ABSTRACT OF BIDS - CONSTRUCTION PAGE OF PAGES

INVITATION FOR BIDS NUMBER: DACW25-83-B-0069

DATE ISSUED: 18 July 1983

DATE OPENED: 15 August 1983

ISSUING OFFICE: US Army Engineer District, Rock Island, Corps of Engineers, Clock Tower Building, Rock Island, IL 61201

FOR: Lock Rehabilitation, Lockport Lock, Stage I, Illinois Waterway

NUMBER OF ADDENDA ISSUED: 0001, 0002, 0003, 0004, 0005

GOVERNMENT ESTIMATE:
- [ ] HIRED LABOR ESTIMATE
- [X] REASONABLE CONTRACT ESTIMATE (without profit)
- [ ] REASONABLE CONTRACT ESTIMATE (including profit)

CERTIFICATE

I CERTIFY that I have opened, read, and recorded herein all bids received in response to this invitation.

R. H. REESINK — SIGNATURE Chief, Proc & Supply Div.

BID NO.	4 (Lg)	1 (Lg)	2 (Lg)
	J. A. Jones Const. Co. 6060 St. Albans St. Charlotte, NC 28287	Morrison-Knudsen Co, Inc. PO Box 7808 Boise, Idaho 83729	Traylor Bros, Inc. 835 North Congress Ave Evansville, IN 47715
BID SECURITY	20% Bid Bond	20% Bid Bond	20% Bid Bond
ADDENDA ACKNOWLEDGED	Five Amendments	Five Amendments	Five Amendments

ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
1.	Protection and Unwatering	1	JOB	SUM	168,800.00	SUM	155,000.00	SUM	500,000.00	SUM	1,200,000.00
2.	Site Preparation										
a.	Temporary Field Office	1	JOB	SUM	25,900.00	SUM	47,000.00	SUM	45,000.00	SUM	40,000.00
b.	Removal & Replacement of Existing Security Fencing	1	JOB	SUM	5,100.00	SUM	5,900.00	SUM	12,000.00	SUM	30,000.00
c.	Removal & Replacement of Existing Handrails	1	JOB	SUM	9,000.00	SUM	37,000.00	SUM	39,000.00	SUM	40,000.00
d.	Removal & Replacement of Control Houses	1	JOB	SUM	8,000.00	SUM	4,700.00	SUM	15,000.00	SUM	50,000.00
3.	Stabilization of Upper Service Gate Sill										
a.	Mobilization & Demobilization	1	JOB	SUM	38,800.00	SUM	26,000.00	SUM	35,000.00	SUM	50,000.00
b.	Drilling Grout Holes	2,085	L.F.	10.00	20,850.00	22.40	46,704.00	20.00	41,700.00	30.00	62,550.00
c.	Drilling Exploratory Holes	150	L.F.	25.25	3,787.50	31.00	4,650.00	25.00	3,750.00	40.00	6,000.00
d.	Placing Grout	1,475	C.F.	9.00	13,275.00	20.00	29,500.00	26.00	38,350.00	15.00	22.125.00

DD FORM 1 OCT 78 1501-1 EDITION OF 1 NOV 64 IS OBSOLETE

(Continued)

(Sheet 1 of 7)

Table A2 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION		PAGE OF PAGES				BID NO. 4		BID NO. 1		BID NO. 2	
ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUN
3.											
e.	Connections to Grout Holes	128	EACH	5.00	640.00	5.00	640.00	5.00	640.00	5.00	640.00
f.	Portland Cement	660	C.F.	3.25	2,145.00	7.30	4,818.00	5.00	3,300.00	10.00	6,600.00
g.	Sand	1,150	C.F.	2.00	2,300.00	3.20	3,680.00	1.00	1,150.00	3.00	3,450.00
h.	Coarse Aggregate	165	C.F.	2.50	412.50	2.40	396.00	2.00	330.00	3.00	495.00
i.	Placing Dental Treatment	500	C.F.	9.00	4,500.00	5.90	2,950.00	25.00	12,500.00	30.00	15,000.00
j.	Rock Anchors	1,702	L.F.	30.50	51,911.00	28.00	47,656.00	20.00	34,040.00	20.00	34,040.00
k.	Drilling Anchor Holes	1,702	L.F.	15.50	26,381.00	28.00	47,656.00	20.00	34,040.00	60.00	102,120.00
l.	Test Anchors	1	JOB	SUM	1,500.00	SUM	6,500.00	SUM	6,000.00	SUM	20,000.00
m.	Filler Grouting of Anchors	115	C.F.	45.00	5,175.00	37.00	4,255.00	51.00	5,865.00	20.00	2,300.00
n.	Concrete Removal	1,375	C.F.	36.00	49,500.00	22.80	31,350.00	28.00	38,500.00	35.00	48,125.00
o.	Steel Reinforcement	3,300	LB	1.00	3,300.00	1.59	5,247.00	1.50	4,950.00	0.80	2,640.00
p.	Concrete	51	C.Y.	425.00	21,675.00	146.00	7,446.00	1500.00	76,500.00	500.00	25,500.00
4.	Upper Lift Gate Modifications, Roller Assembly	1	JOB	SUM	85,400.00	SUM	100,000.00	SUM	250,000.00	SUM	200,000.00
5.	Upper Lift Gate Recess Modifications										
a.	Concrete Removal	6,260	C.F.	47.00	294,220.00	18.30	114,558.00	27.00	169,020.00	35.00	219,100.00
b.	Concrete Anchors	906	EACH	54.00	48,924.00	46.00	41,676.00	31.00	28,086.00	50.00	45,300.00
c.	Steel Reinforcement	10,100	LB	1.10	11,110.00	0.60	6,060.00	1.50	15,150.00	0.80	8,080.00
d.	Concrete	75	C.Y.	546.00	40,950.00	1502.00	112,650.00	1450.00	108,750.00	700.00	52,500.00
e.	Lift Gate Bearing Plate Assembly	43,600	LB	3.75	163,500.00	1.55	67,580.00	3.50	152,600.00	3.00	130,800.00
f.	Lifting Beam Bearing Plates	1,130	LB	3.75	4,237.50	2.50	2,825.00	5.00	5,650.00	6.00	6,780.00

(Continued)

(Sheet 2 of 7)

Table A2 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION		PAGE OF PAGES				BID NO. 4		BID NO. ; 1		BID NO. 2	
ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
g.	Government Furnished Armor	33,100	LB	2.00	66,200.00	1.18	39,058.00	1.00	33,100.00	3.00	99,300.00
h.	Contractor Furnished Armor	21,600	LB	3.25	70,200.00	1.23	26,568.00	3.00	64,800.00	3.50	75,600.00
6.	Lower Gate Bay Resurfacing										
a.	Concrete Removal	9,050	C.F.	46.50	420,825.00	16.80	152,040.00	10.00	90,500.00	35.00	316,750.00
b.	Concrete Anchors	960	EACH	54.00	51,840.00	74.00	71,040.00	33.00	31,680.00	50.00	48,000.00
c.	Steel Reinforcement	19,340	LB	1.10	21,274.00	0.55	10,637.00	1.50	29,010.00	0.80	15,472.00
d.	Concrete	245	C.Y.	546.00	133,770.00	861.00	210,945.00	750.00	183,750.00	900.00	220,500.00
e.	Pressure Grouting	22	C.F.	624.50	13,739.00	112.00	2,464.00	825.00	18,150.00	100.00	2,200.00
f.	Government Furnished Armor	25,700	LB	2.00	51,400.00	1.28	32,896.00	0.75	19,275.00	3.00	77,100.00
g.	Contractor Furnished Armor	12,750	LB	3.25	41,437.50	1.22	15,555.00	2.50	31,875.00	3.50	44,625.00
7.	New Cable Trench										
a.	Concrete, Rock, and Conduit Removal	1	JOB	SUM	40,800.00	SUM	11,700.00	SUM	50,000.00	SUM	10,000.00
b.	Conduits	2,034	L.F.	4.00	8,136.00	3.50	7,119.00	10.00	20,340.00	10.00	20,340.00
c.	Concrete Fill	1	JOB	SUM	23,500.00	SUM	14,500.00	SUM	27,000.00	SUM	8,000.00
d.	Manholes	2	EACH	7710.00	15,420.00	965.00	1,930.00	13000.00	26,000.00	15000.00	30,000.00
8.	Removal of Miter Gates, Complete with Appurtenant Parts	1	JOB	SUM	239,400.00	SUM	500,000.00	SUM	900,000.00	SUM	500,000.00
9.	New Miter Gates	1	JOB	SUM	1,510,300.00	SUM	2,862,000.00	SUM	2,450,000.00	SUM	2,000,000.00
10.	Miter Gate Quoin Modifications										
a.	Concrete Anchors	66	EACH	54.00	3,564.00	38.00	2,508.00	44.00	2,904.00	50.00	3,300.00
b.	Concrete	74	C.Y.	546.00	40,404.00	792.00	58,608.00	345.00	25,530.00	700.00	51,800.00

(Continued)

(Sheet 3 of 7)

Table A2 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION | PAGE 4 OF 7 PAGES

ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	BID NO. 4 UNIT PRICE	BID NO. 4 ESTIMATED AMOUNT	BID NO. 1 UNIT PRICE	BID NO. 1 ESTIMATED AMOUNT	BID NO. 2 UNIT PRICE	BID NO. 2 ESTIMATED AMOUNT
c.	Embedded Metals	37,800	LB	3.25	122,850.00	2.10	79,380.00	3.00	113,400.00	3.00	113,400.00
11.	Miter Gate Anchorage Modifications										
a.	Concrete Removal	8,325	C.F.	46.50	387,112.50	15.20	126,540.00	36.00	299,700.00	35.00	291,375.00
b.	Concrete Anchors	524	EACH	54.00	28,296.00	40.00	20,960.00	33.00	17,292.00	50.00	26,200.00
c.	Steel Reinforcement	10,200	LB	1.10	11,220.00	0.84	8,568.00	1.50	15,300.00	0.80	8,160.00
d.	Concrete	310	C.Y.	546.00	169,260.00	728.00	225,680.00	500.00	155,000.00	500.00	155,000.00
e.	Anchorage Links	28,150	LB	3.60	101,340.00	1.46	41,099.00	7.50	211,125.00	6.00	168,900.00
f.	Anchorage Casting Testing	1	JOB	SUM	1,600.00	SUM	3,500.00	SUM	5,000.00	SUM	5,000.00
g.	Anchorage Casting	1	JOB	SUM	1,500.00	SUM	6,350.00	SUM	32,000.00	SUM	20,000.00
12.	Miter Gate Sill Modifications										
a.	Concrete Removal	8,000	C.F.	46.50	372,000.00	13.95	111,600.00	24.00	192,000.00	25.00	200,000.00
b.	Mobilization & Demobilization	1	JOB	SUM	11,000.00	SUM	13,000.00	SUM	12,000.00	SUM	20,000.00
c.	Drilling Grout Holes	312	L.F.	10.00	3,120.00	21.00	6,552.00	20.00	6,240.00	30.00	9,360.00
d.	Drilling Exploratory Holes	120	L.F.	25.25	3,030.00	31.00	3,720.00	25.00	3,000.00	40.00	4,800.00
e.	Placing Grout	315	C.F.	9.50	2,992.50	20.40	6,426.00	26.00	8,190.00	15.00	4,725.00
f.	Connections to Grout Holes	52	EACH	$ 5.00	260.00	5.00	260.00	5.00	260.00	5.00	260.00
g.	Portland Cement	150	C.F.	3.25	487.50	7.28	1,092.00	5.00	750.00	10.00	1,500.00
h.	Sand	255	C.F.	2.00	510.00	3.20	816.00	1.00	255.00	3.00	765.00
i.	Coarse Aggregate	45	C.F.	2.50	112.50	2.40	108.00	2.00	90.00	3.00	135.00
j.	Placing Dental Treatment	135	C.F.	9.50	1,282.50	6.00	810.00	25.00	3,375.00	30.00	4,050.00

(Continued)

(Sheet 4 of 7)

Table A2 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION | PAGE OF PAGES

ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	BID NO. 4 UNIT PRICE	BID NO. 4 ESTIMATED AMOUNT	BID NO. 1 UNIT PRICE	BID NO. 1 ESTIMATED AMOUNT	BID NO. 2 UNIT PRICE	BID NO. 2 ESTIMATED AMOUNT
k.	Rock Anchors	1,377	L.F.	17.50	24,097.50	13.00	17,901.00	10.00	13,770.00	10.00	13,770.00
l.	Drilling Anchor Holes	1,244	L.F.	10.00	12,440.00	14.50	18,038.00	20.00	24,880.00	40.00	49,760.00
m.	Test Anchors	1	JOB	SUM	1,000.00	SUM	3,200.00	SUM	2,500.00	SUM	5,000.00
n.	Filler Grouting of Anchors	15	C.F.	48.00	720.00	74.00	1,110.00	51.00	765.00	20.00	300.00
o.	Steel Reinforcement	17,300	LB	1.10	19,030.00	0.78	13,494.00	1.50	25,950.00	0.80	13,840.00
p.	Concrete	250	C.Y.	546.00	136,500.00	174.00	43,500.00	320.00	80,000.00	800.00	200,000.00
q.	Embedded Metals	11,720	LB	3.25	38,090.00	3.55	41,606.00	4.00	46,880.00	5.00	58,600.00
13.	Field Erection of Miter Gates	1	JOB	SUM	1,276,822.50	SUM	1,140,000.00	SUM	1,582,000.00	SUM	1,300,000.00
14.	Removal of Miter Gate Operating Equipment	1	JOB	SUM	73,400.00	SUM	12,500.00	SUM	30,000.00	SUM	30,000.00
15.	New Miter Gate Operating Equipment	1	JOB	SUM	216,400.00	SUM	160,000.00	SUM	270,000.00	SUM	200,000.00
16.	Miter Gate Operating Equipment Pit Modifications										
a.	Concrete Removal	830	C.F.	46.50	38,595.00	13.30	11,039.00	22.00	18,260.00	35.00	29,050.00
b.	Concrete Anchors	220	EACH	54.00	11,880.00	39.20	8,624.00	33.00	7,260.00	50.00	11,000.00
c.	Steel Reinforcement	1,960	LB	1.10	2,156.00	0.70	1,372.00	1.50	2,940.00	0.80	1,568.00
d.	Concrete	1	JOB	SUM	12,600.00	SUM	29,600.00	SUM	10,000.00	SUM	20,000.00
e.	Machinery Support	21,350	LB	2.25	48,037.50	2.08	44,408.00	3.50	74,725.00	2.00	42,700.00
f.	Miscellaneous Structural Steel Metals	5,200	LB	1.50	7,800.00	2.12	11,024.00	4.00	20,800.00	5.00	26,000.00
g.	Miscellaneous Aluminum Metals	2,900	LB	6.50	18,850.00	5.35	15,515.00	3.50	10,150.00	5.00	14,500.00
h.	Cast Iron Frame	120	L.F.	25.00	3,000.00	37.70	4,524.00	50.00	6,000.00	100.00	12,000.00
i.	Modify Existing Bull Gear, Rollers, and Strut Arms	1	JOB	SUM	34,300.00	SUM	24,300.00	SUM	150,000.00	SUM	100,000.00

(Continued)

(Sheet 5 of 7)

Table A2 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION				PAGE 6 OF 7 PAGES		BID NO. 4		BID NO. 1		BID NO. 2	
ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
17.	Relocate Power	1	JOB	SUM	18,000.00	SUM	200,000.00	SUM	175,000.00	SUM	160,000.00
18.	Ladders & Protective Armor										
a.	Concrete Removal	4,120	C.F.	46.50	191,580.00	17.65	72,718.00	15.00	61,800.00	35.00	144,200.00
b.	Concrete Anchors	840	EACH	54.00	45,360.00	39.80	33,432.00	33.00	27,720.00	50.00	42,000.00
c.	Steel Reinforcement	9,160	LB	1.10	10,076.00	0.80	7,328.00	1.50	13,740.00	0.80	7,328.00
d.	Concrete	150	C.Y.	546.00	81,900.00	940.00	141,000.00	750.00	112,500.00	800.00	120,000.00
e.	Government Furnished Armor	40,650	LB	2.00	81,300.00	1.20	48,780.00	1.00	40,650.00	3.00	121,950.00
f.	Contractor Furnished Armor	28,250	LB	3.25	91,812.50	1.38	38,985.00	2.50	70,625.00	3.50	98,875.00
g.	Miscellaneous Metals	6,500	LB	3.25	21,125.00	1.93	12,545.00	3.00	19,500.00	7.00	45,500.00
19.	Floating Mooring Bitt Armor Sta. 62+78 & Sta. 61+01										
a.	Concrete Removal	1,865	C.F.	46.50	86,722.50	18.60	34,689.00	30.00	55,950.00	35.00	65,275.00
b.	Concrete Anchors	312	EACH	54.00	16,848.00	39.00	12,168.00	31.00	9,672.00	50.00	15,600.00
c.	Steel Reinforcement	4,740	LB	1.10	5,214.00	1.00	4,740.00	1.50	7,110.00	0.80	3,792.00
d.	Concrete	70	C.Y.	546.00	38,220.00	954.00	66,780.00	1475.00	103,250.00	800.00	56,000.00
e.	Government Furnished Armor	20,400	LB	2.00	40,800.00	1.29	26,316.00	4.00	81,600.00	3.00	61,200.00
f.	Contractor Furnished Armor	12,450	LB	3.25	40,462.50	1.14	14,193.00	1.50	18,675.00	3.50	43,575.00
20.	Floating Mooring Bitt Armor Sta. 66+63										
a.	Concrete Removal	1,050	C.F.	46.50	48,825.00	17.00	17,850.00	28.00	29,400.00	35.00	36,750.00
b.	Concrete Anchors	158	EACH	54.00	8,532.00	39.00	6,162.00	31.00	4,898.00	50.00	7,900.00
c.	Steel Reinforcement	2,500	LB	1.10	2,750.00	1.00	2,500.00	1.50	3,750.00	0.80	2,000.00

(Continued)

(Sheet 6 of 7)

Table A2 (Concluded)

ABSTRACT OF BIDS - CONSTRUCTION	PAGE 7 OF 7 PAGES					BID NO. 4		BID NO. 1		BID NO. 2	
ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
d.	Concrete	40	C.Y.	546.00	21,840.00	855.00	34,200.00	1450.00	58,000.00	800.00	32,000.00
e.	Government Furnished Armor	11,500	LB	2.00	23,000.00	1.23	14,145.00	4.00	46,000.00	3.00	34,500.00
f.	Contractor Furnished Armor	6,350	LB	3.25	20,637.50	1.10	6,985.00	2.00	12,700.00	3.50	22,225.00
21.	Removal of Shutter Gate	1	JOB	SUM	47,800.00	SUM	49,100.00	SUM	37,400.00	SUM	20,000.00
	TOTAL OF ESTIMATED AMOUNTS (ITEMS 1 THRU 21, INCL)				8,000,000.00		8,086,797.00		10,122,582.00		10,249,520.00
	TIME FOR ACCEPTANCE OF BIDS						30 Days		30 Days		30 Days
	3 - Kenny Construction Co, Illinois Constructors Corp., (Lg)										
	& Thomas M. Madden Co. A Joint Venture										
	250 Northgate Parkway										
	Wheeling, Illinois 60090										
	$10,381,051.00										

(Sheet 7 of 7)

Table A3

Abstract of Bids for Stage I Rehabilitation, Brandon Road Lock

ABSTRACT OF BIDS - CONSTRUCTION PAGE OF PAGES

INVITATION FOR BIDS NUMBER: DACW25-84-B-0007

DATE ISSUED: 12 December 1983

DATE OPENED: 24 January 1984

ISSUING OFFICE: US Army Engineer District, Rock Island
Corps of Engineers
Clock Tower Building
Rock Island, Illinois 61201

FOR: Lock Rehabilitation, Brandon Road Lock - Stage I, Illinois Waterway

NUMBER OF ADDENDA ISSUED: 0001, 0002, 0003

GOVERNMENT ESTIMATE:
- [ ] HIRED LABOR ESTIMATE
- XX REASONABLE CONTRACT ESTIMATE (without profit)
- [ ] REASONABLE CONTRACT ESTIMATE (including profit)

CERTIFICATE

I CERTIFY that I have opened, read, and recorded herein all bids received in response to this invitation.

R. H. REESINK SIGNATURE Chief, Proc & Supply Div.

BID NO.	11 (LB)		8 (LB)		5 (LB)	
	James McHugh Const. Co. 2222 S. Indiana Ave. Chicago, IL 60616		S. A. Healy Co. Box 11 McCook, IL 60525		Massman Const. Co. 8901 State Line PO Box 8458 Kansas City, MO 64114	
BID SECURITY	20% Bid Bond		20% Bid Bond		20% Bid Bond	
ADDENDA ACKNOWLEDGED	0001, 0002, 0003		0001, 0002, 0003		0001, 0002, 0003	

ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
1.	Protection and Unwatering	1	JOB	SUM	367,412.00	SUM	500,000.00	SUM	100,000.00	SUM	755,000.00
2.	Removal & Replacement of Existing Control Houses & Miscellaneous Lock Features	1	JOB	SUM	52,621.00	SUM	16,000.00	SUM	50,000.00	SUM	25,000.00
3.	Removal & Replacement of Existing Railing & Security Fencing	1	JOB	SUM	67,242.00	SUM	35,000.00	SUM	15,000.00	SUM	40,000.00
4.	Upper Guide Wall Resurfacing										
a.	Concrete Removal	6,000	C.F.	28.53	171,180.00	16.00	96,000.00	35.00	210,000.00	30.00	180,000.00
b.	Concrete Anchors	460	EACH	43.90	20,194.00	17.00	7,820.00	20.00	9,200.00	60.00	27,600.00
c.	Steel Bar Reinforcement	16,300	LB	1.19	19,397.00	0.50	8,150.00	0.36	5,868.00	1.25	20,375.00
d.	Concrete, Type B	17	C.Y.	284.00	4,828.00	750.00	12,750.00	600.00	10,200.00	1100.00	18,700.00
e.	Government Furnished Armor	33,100	LB.	2.77	91,687.00	1.30	43,030.00	0.75	24,825.00	3.40	112,540.00
f.	Contractor Furnished Armor	57,600	LB.	3.68	211,968.00	1.50	86,400.00	0.85	48,960.00	1.25	72,000.00

DD FORM 1 OCT 75 1501-1 EDITION OF 1 NOV 64 IS OBSOLETE *U.S. Government Printing Office: 1969—341-048/8159

(Continued)

(Sheet 1 of 6)

Table A3 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION		PAGE OF PAGES				BID NO. 11		BID NO. 8		BID NO. 5	
ITEM NO	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
4.											
g.	Concrete, Type C	205	C.Y.	718.20	147,231.00	400.00	82,000.00	340.00	69,700.00	900.00	184,500.00
h.	Check Posts	9	EACH	1591.00	14,319.00	1400.00	12,600.00	800.00	7,200.00	2000.00	18,000.00
5.	Upper Gate Bays Resurfacing										
a.	Concrete Removal	6,650	C.F.	28.50	189,525.00	13.00	86,450.00	40.00	266,000.00	15.00	99,750.00
b.	Concrete Anchors	590	EACH	44.00	25,960.00	17.00	10,030.00	15.00	8,850.00	60.00	35,400.00
c.	Steel Bar Reinforcement	17,900	LB.	1.19	21,301.00	0.50	8,950.00	0.36	6,444.00	1.25	22,375.00
d.	Concrete, Type C	240	C.Y.	718.30	172,392.00	275.00	66,000.00	300.00	72,000.00	1100.00	264,000.00
e.	Concrete, Type D	60	C.Y.	205.75	12,345.00	150.00	9,000.00	375.00	22,500.00	200.00	12,000.00
f.	Government Furnished Armor	19,300	LB.	2.77	53,461.00	1.00	19,300.00	0.50	9,650.00	1.25	24,125.00
g.	Contractor Furnished	14,900	LB.	3.59	53,491.00	1.70	25,330.00	1.00	14,900.00	1.70	25,330.00
6.	Lock Chamber Resurfacing										
a.	Concrete Removal, Including Sidewalk	72,250	C.F.	28.54	2,062,015.00	6.00	433,500.00	30.00	2,167,500.00	6.00	433,500.00
b.	Concrete Anchors	3,785	EACH	44.00	166,540.00	15.00	56,775.00	15.00	56,775.00	60.00	227,100.00
c.	Steel Bar Reinforcement	174,100	LB.	1.19	207,179.00	0.50	87,050.00	0.36	62,676.00	1.25	217,625.00
d.	Steel Wire Fabric Reinforcement	2,700	S.F.	0.57	1,539.00	0.35	945.00	0.30	810.00	2.00	5,400.00
e.	Concrete Type C	2,820	C.Y.	718.20	2,025,324.00	230.00	648,600.00	275.00	775,500.00	375.00	1,057,500.00
f.	Concrete, Type D	850	C.Y.	205.72	174,862.00	120.00	102,000.00	200.00	170,000.00	100.00	85,000.00
g.	Check Posts	18	EACH	1591.00	28,638.00	1400.00	25,200.00	800.00	14,400.00	2000.00	36,000.00
h.	Government Furnished Armor	258,100	LB.	2.51	647,831.00	1.00	258,100.00	0.50	129,050.00	1.25	322,625.00
i.	Contractor Furnished Armor	121,900	LB.	3.68	448,592.00	1.50	182,850.00	1.00	121,900.00	1.20	146,280.00

(Continued)

(Sheet 2 of 6)

Table A3 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION		PAGE OF PAGES				BID NO. 11		BID NO. 8		BID NO. 5	
ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
j.	Line Hooks	30	EACH	435.00	13,050.00	1400.00	42,000.00	600.00	18,000.00	2500.00	75,000.00
k.	Ladders, Lock Face	4	EACH	6092.50	24,370.00	7500.00	30,000.00	3000.00	12,000.00	5000.00	20,000.00
l.	Ladders, Cablewell and Manhole	1	JOB	SUM	14,542.00	SUM	20,000.00	SUM	115,000.00	SUM	25,000.00
m.	Remove and Replace Roadway	1	JOB	SUM	20,504.00	SUM	3,500.00	SUM	10,000.00	SUM	5,000.00
n.	Air Vent Covers	1	JOB	SUM	9,987.00	SUM	2,000.00	SUM	5,000.00	SUM	10,000.00
7.	Lower Gate Bays Resurfacing										
a.	Concrete Removal	19,500	C.F.	28.53	556,335.00	8.00	156,000.00	30.00	585,000.00	10.00	195,000.00
b.	Concrete Anchors	860	EACH	43.90	37,754.00	15.00	12,900.00	15.00	12,900.00	60.00	51,600.00
c.	Steel Bar Reinforcement	40,000	LB.	1.19	47,600.00	0.50	20,000.00	0.36	14,400.00	1.25	50,000.00
d.	Concrete, Type C	750	C.Y.	718.20	538,650.00	200.00	150,000.00	200.00	150,000.00	500.00	375,000.00
e.	Concrete, Type D	75	C.Y.	205.72	15,429.00	120.00	9,000.00	50.00	3,750.00	200.00	15,000.00
f.	Check Posts	1	EACH	1592.00	1,592.00	1500.00	1,500.00	800.00	800.00	2000.00	2,000.00
g.	Government Furnished Armor	40,120	LB.	2.75	110,330.00	1.00	40,120.00	0.50	20,060.00	1.25	50,150.00
h.	Contractor Furnished Armor	19,700	LB.	3.68	72,496.00	1.50	29,550.00	0.85	16,745.00	1.70	33,490.00
i.	FMB Framing and Roller Guide Assembly	1	JOB	SUM	18,527.00	SUM	15,000.00	SUM	50,000.00	SUM	15,000.00
8.	Lower Gate Forebays Resurfacing										
a.	Concrete Removal	1,720	C.F.	28.50	49,020.00	25.00	43,000.00	20.00	34,400.00	10.00	17,200.00
b.	Concrete Anchors	102	EACH	44.00	4,488.00	17.00	1,734.00	15.00	1,530.00	60.00	6,120.00
c.	Steel Bar Reinforcement	3,900	LB.	1.19	4,641.00	0.60	2,340.00	0.36	1,404.00	1.25	4,875.00
d.	Concrete, Type C	65	C.Y.	711.00	46,215.00	250.00	16,250.00	375.00	24,375.00	600.00	39,000.00

(Continued)

(Sheet 3 of 6)

Table A3 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION		PAGE		OF	PAGES	BID NO. 11		BID NO. 8		BID NO. 5	
ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
8. e.	Government Furnished Armor	18,250	LB.	2.78	50,735.00	1.00	18,250.00	0.50	9,125.00	1.00	18,250.00
f.	Contractor Furnished Armor and Buffer Channel	13,900	LB.	2.84	39,476.00	1.50	20,850.00	0.85	11,815.00	1.70	23,630.00
g.	Deleted										
h.	Bulkhead Recess Protection Plates	10,650	LB.	4.60	48,990.00	1.80	19,170.00	2.00	21,300.00	2.00	21,300.00
9.	Stairway Resurfacing										
a.	Concrete Removal	660	C.F.	31.50	20,790.00	10.00	6,600.00	10.00	6,600.00	30.00	19,800.00
b.	Concrete Anchors	170	EACH	38.30	6,511.00	17.00	2,890.00	25.00	4,250.00	60.00	10,200.00
c.	Steel Bar Reinforcement	1,570	LB.	1.00	1,570.00	0.70	1,099.00	0.40	628.00	1.50	2,355.00
d.	Concrete, Type C	25	C.Y.	304.88	7,622.00	425.00	10,625.00	100.00	2,500.00	1000.00	25,000.00
10.	Stabilization & Resurfacing of Lower Guide Wall										
a.	Concrete Removal	29,600	C.F.	28.53	844,488.00	7.00	207,200.00	10.00	296,000.00	6.00	177,600.00
b.	Drilling Anchor Holes	3,400	L.F.	29.88	101,592.00	20.00	68,000.00	20.00	68,000.00	30.00	102,000.00
c.	Rock Anchors	3,500	L.F.	20.97	73,395.00	20.00	70,000.00	18.00	63,000.00	25.00	87,500.00
d.	Test Anchors	1	JOB	SUM	8,542.00	SUM	20,000.00	SUM	7,000.00	SUM	25,000.00
e.	Grouting of Anchors	450	C.F.	68.30	30,735.00	45.00	20,250.00	40.00	18,000.00	75.00	33,750.00
f.	Steel Bar Reinforcement	63,000	LB.	1.19	74,970.00	0.50	31,500.00	0.36	22,680.00	1.25	78,750.00
g.	Concrete, Type A	90	C.Y.	126.60	11,394.00	220.00	19,800.00	150.00	13,500.00	700.00	63,000.00
h.	Concrete, Type C	1,150	C.Y.	718.20	825,930.00	190.00	218,500.00	90.00	103,500.00	425.00	488,750.00
i.	Government Furnished Armor	110,960	LB.	2.75	305,140.00	1.00	110,960.00	0.40	44,384.00	1.25	138,700.00
j.	Contractor Furnished Armor	65,750	LB.	3.68	241,960.00	1.50	98,625.00	0.90	59,175.00	1.20	78,900.00

(Continued)

(Sheet 4 of 6)

Table A3 (Continued)

ABSTRACT OF BIDS - CONSTRUCTION		PAGE 5 OF 6 PAGES				BID NO. 11		BID NO. 8		BID NO. 5	
ITEM NO.	DESCRIPTION OF BID ITEM	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT	UNIT PRICE	ESTIMATED AMOUNT
10. k.	Check Posts	5	EACH	2275.00	11,375.00	1300.00	6,500.00	700.00	3,500.00	2000.00	10,000.00
l.	Lockwall Subdrain	572	L.F.	59.25	33,891.00	0.50	28,600.00	18.00	10,296.00	100.00	57,200.00
m.	Concrete Anchors	960	EACH	43.90	42,144.00	17.00	16,320.00	10.00	9,600.00	60.00	57,600.00
11.	Lower Guide Wall Cell Replacement										
a.	Concrete Removal	4,700	C.F.	28.53	134,091.00	2.00	9,400.00	10.00	47,000.00	6.00	28,200.00
b.	Removal of Overburden and Final Gradeing	1	JOB	SUM	12,014.00	SUM	11,000.00	SUM	5,000.00	SUM	15,000.00
c.	Steel Sheet Piling	1,710	S.F.	24.90	42,579.00	22.00	37,620.00	20.00	34,200.00	43.00	73,530.00
d.	Concrete, Type C	540	C.Y.	135.75	73,305.00	75.00	40,500.00	50.00	27,000.00	100.00	54,000.00
e.	Metal Work	1	JOB	SUM	31,087.00	SUM	35,000.00	SUM	15,000.00	SUM	25,000.00
f.	Check Posts	2	EACH	1097.50	2,195.00	1400.00	2,800.00	500.00	1,000.00	2000.00	4,000.00
g.	New Fence	1	JOB	SUM	1,609.00	SUM	1,000.00	SUM	1,000.00	SUM	2,500.00
12.	Rehabilitation of Upper Guard and Both Service Gates										
a.	Replace Rivets with Approved Fasteners										
(1)	First 8,000	8,000	EACH	16.75	134,000.00	13.00	104,000.00	20.00	160,000.00	35.00	280,000.00
(2)	Over 8,000	1,000	EACH	16.94	16,940.00	11.00	11,000.00	20.00	20,000.00	20.00	20,000.00
b.	All Other Work	1	JOB	SUM	1,512,536.00	SUM	2,275,000.00	SUM	1,954,675.00	SUM	2,217,500.00
13.	Deleted										
14.	Existing House Modification	1	JOB	SUM	12,119.00	SUM	25,000.00	SUM	225,000.00	SUM	25,000.00
15.	Rewiring and Relighting of Lock	1	JOB	SUM	1,001,671.00	SUM	900,000.00	SUM	300,000.00	SUM	670,000.00
	TOTAL OF ESTIMATED AMOUNTS (ITEMS 1 THRU 15, INCL)				14,800,000.00		7,962,783.00		9,090,000.00		10,391,175.00
	TIME FOR ACCEPTANCE OF BIDS						30 Days		30 Days		30 Days

(Continued)

(Sheet 5 of 6)

Table A3 (Concluded)

4 - Kenny Construction Co. (LB) 250 Northgate Parkway Wheeling, IL 60090 $10,671,486.00	1 - Traylor Bros., Inc. (LB) PO Box 5165 Evansville, IN 47715 $10,688,010.00	7 - Al Johnson Construction Co. (LB) 3209 West 76th St. Minneapolis, MN 55435 $10,758,160.00
3 - Morrison-Knudsen Co., Inc (LB) PO Box 7808 Boise, Idaho 83729 $10,909,681.00	9 - Thos M. Madden Co (SB) Illinois Constructors Corp (JV) 6400 South East Avenue Hodgkins, IL 60525 $11,659,744.00	10 - J. A. Jones Construction Co. (LB) 6060 St. Albans St. Charlotte, NC $11,767,771.00
2 - Kiewit Eastern Co. (LB) Suite A1, Rivers Center 10260 Old Columbia Road Columbia, MD 21046 $12,161,380.00	6 - Gust K. Newberg Construction Co. (LB) 2040 N. Ashland Chiçago, IL 60614 $12,257,000.00	

(Sheet 6 of 6)